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Journal

VOL. 51 • NO. 1

AMERICAN WATER WORKS ASSOCIATION

In this issue:

Summer in the Canadian North, where water problems are always on ice

Journal

AMERICAN WATER WORKS ASSOCIATION

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Coming Meetings

AWWA SECTIONS

Winter-Spring 1959

Jan. 27—New York Section, Midwinter Luncheon Meeting, at Park Sheraton Hotel, New York. Secretary, Kimball Blanchard, 2222 Jackson Ave., Long Island City 1.

Feb. 4–6—Indiana Section, at Sheraton-French Lick Hotel, French Lick. Secretary, Chester H. Canham, State Board of Health, 1330 W. Michigan St., Indianapolis 7.

Mar. 11–13—Illinois Section, at Morrison Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago 1.

Mar. 19—New England Section, at Statler Hotel, Boston, Mass. Secretary, J. E. Revelle, Dist. Sales Mgr., Chicago Bridge & Iron Co., 201 Devonshire St., Boston 10, Mass.

Apr. 5–8—Southeastern Section, at Wade Hampton Hotel, Columbia, S.C. Secretary, N. M. deJarnette, Engr., Water Quality Div., State Dept. of Public Health, 309 State Office Bldg., Atlanta 3, Ga.

Apr. 8–10—New York Section, at Powers Hotel, Rochester. Secretary, Kimball Blanchard, New York Branch Sales Office, Neptune Meter Co., 2222 Jackson Ave., Long Island City 1.

Apr. 9–11—Montana Section, at Jordan Hotel, Glendive. Secretary, Arthur W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

Apr. 15–17—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, Rupert C. Ott Jr., Repr., Neptune Meter Co., 2818—21st St., Columbus.

Apr. 16–18—Arizona Section, at Hi-Way House Hotel, Phoenix. Secretary, Stanford I. Roth, Supvr. of Water Collections, Div. of Water & Sewers, Phoenix.

Apr. 22–24—Kansas Section, at Besse Hotel, Pittsburg. Secretary, Harry W. Badley, Repr., Neptune Meter Co., 119 W. Cloud, Salina.

Apr. 23–25—Pacific Northwest Section, at Vancouver Hotel, Vancouver, B.C. Secretary, Fred D. Jones, W. 2108 Maxwell Ave., Spokane 11, Wash.

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Coming Meetings

May 3–6—Canadian Section, at Queen Elizabeth Hotel, Montreal, Que. Secretary, A. E. Berry, Gen. Mgr. & Chief Engr., Ontario Water Resources Com., Parliament Bldgs., Toronto, Ont.

Jun. 3–5—Pennsylvania Section, at Salem House, Wernersville. Secretary, L. S. Morgan, Chief, Mine Drainage Sec., 413 First National Bank Bldg., Greensburg.

Fall 1959

Sep. 9–11—Wisconsin Sec., Milwaukee.

Sep. 14–16—Kentucky-Tennessee Sec., Lexington, Ky.

Sep. 16–18—New York Sec., Upper Saranac Lake.

Sep. 23–25—Michigan Sec., Saginaw.

Sep. 27–29—Missouri Sec., Kansas City.

Oct. 7–9—Chesapeake Sec.

Oct. 14–16—Iowa Sec., Des Moines.

Oct. 18–21—Alabama-Mississippi Sec., New Orleans, La.

Oct. 18–21—Southwest Sec., New Orleans, La.

Oct. 22–24—New Jersey Sec., Atlantic City.

Oct. 28–29—West Virginia Sec., Parkersburg.

Oct. 28–30—Ohio Sec., Dayton.

Oct. 30—California Sec., Bakersfield.

Nov. 4–6—Virginia Sec., Roanoke.

Nov. 9–11—North Carolina Sec., Durham.

Nov. 15–19—Florida Sec., Tampa.

OTHER ORGANIZATIONS

Jan. 19–23—American Institute of Electrical Engineers, New York, N.Y.

Jan. 26–28—Seminar on Radiological Health, sponsored by Univ. of North Carolina and North Carolina Board of Health, at Chapel Hill, N.C. Write: Prof. E. T. Chanlett, Dept. of Sanitary

(*Continued from page 6*)

Engineering, School of Public Health, Univ. of North Carolina, Chapel Hill, N.C.

Feb. 15–19—American Institute of Mining, Metallurgical & Petroleum Engineers, San Francisco, Calif.

Mar. 1–6—Texas Water & Sewage Works Assn. Short School, Texas A&M College, College Sta., Tex.

Mar. 2–13—Short courses on "Water Quality Management—Sanitary Engineering Aspects" and "Basic Radiological Health," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Apply to USPHS regional office director or to Chief, Training Program, R. A. Taft Sanitary Engineering Center, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

Mar. 16–25—Short course on "Environmental Health Aspects of Nuclear Reactor Operations," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Apply as above.

Mar. 16–20—National Assn. of Corrosion Engineers, Sherman Hotel, Chicago, Ill.

Mar. 16–20—Western Metal Exposition & Congress, sponsored by American Society for Metals and other technical groups, Pan-Pacific Auditorium and Ambassador Hotel, Los Angeles, Calif. Write: Ray T. Bayless, Asst. Secy., 7301 Euclid Ave., Cleveland 3, Ohio.

Apr. 5–10—5th Nuclear Congress, Public Auditorium, Cleveland, Ohio, including Nuclear Engineering & Science Conference, sponsored by AWWA and other societies; Atomic Energy Management Conference, sponsored by Atomic Industrial Forum and National Industrial Conference Board; Hot Laboratories & Equipment Conference; and Atomfair, sponsored by Atomic Industrial Forum. Write: T. A. Marshall Jr., Mgr., 1959 Nuclear Congress, c/o Engineers Joint Council, 29 W. 39th St., New York 18, N.Y.


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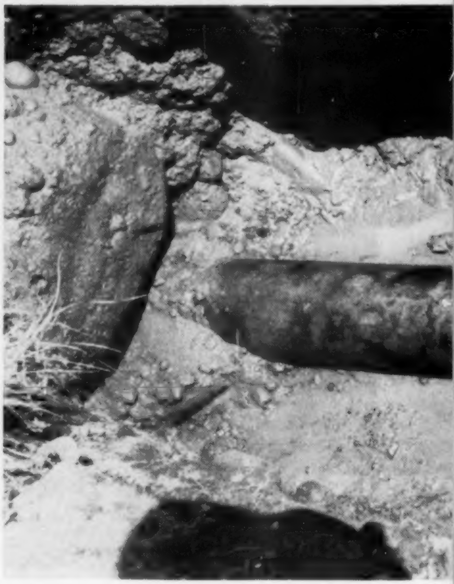

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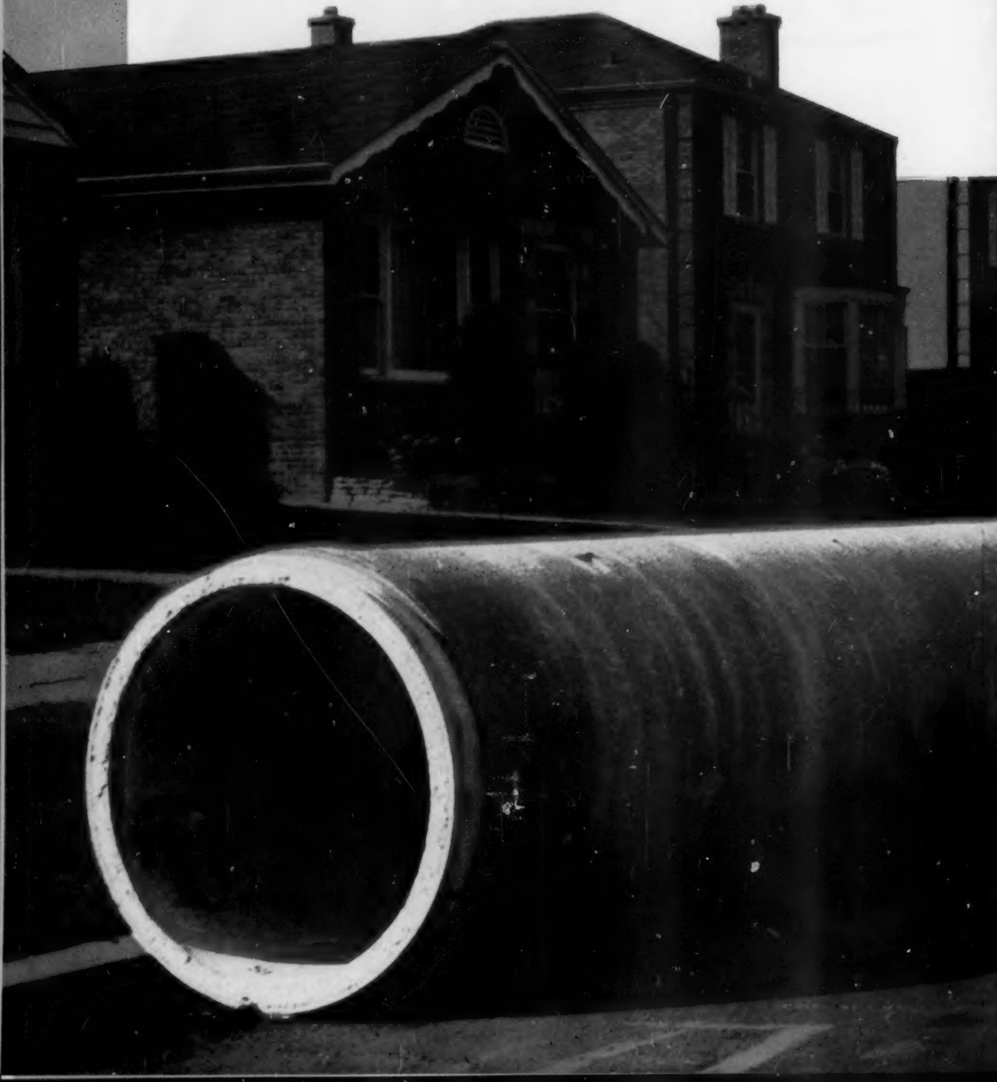

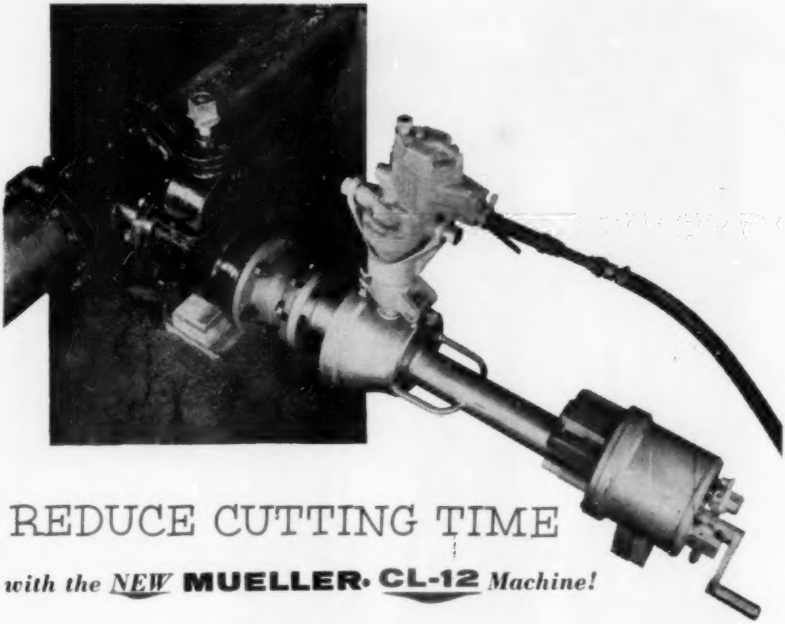

Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 51 • JANUARY 1959 • NO. 1

Penny-Wise Water

Lewis S. Finch

A message to the water utility industry by Lewis S. Finch, President, American Water Works Association.

DURING the past 18 months, the several sections of the American Water Works Association have been told of the importance of making water utility customers conscious of the value to them of the water service they now take for granted. It also has been stressed that, if water utility management seeks public appreciation of the value of water service, it must merit it.

During this time, the Association itself has not been idle. It has developed a Water Works Advancement Program designed to accomplish these goals and has established a Water Works Advancement Committee, which is responsible for the direction of the effort. This committee is responsible only to the Board of Directors. It is headed by Fred Merryfield, past president, in whose administration the project was conceived. James B. Corey, a specialist in public relations work, has been employed to develop a plan of attack and administer the program.

In helping to establish the program, during my tour of duty as vice-president, and since becoming president of the Association, I have been particularly impressed by the fact that many water systems are in need of improvement and expansion, but that those responsible for their management are hesitant to spend the money necessary to do the job properly. They hesitate largely because they are afraid of customer reaction to increased rates. As a consequence, they either do not improve their facilities or, if they do expand, they do it on a hand-to-mouth basis. It was in considering these circumstances that I began thinking in terms of "Penny-Wise Water"—hence this article.

Why Penny Wise?

"Penny wise and pound foolish"—this ancient adage applied to water expresses my theme. To be short of water because it costs too much to build needed improvements is false economy. To build plant facilities too small to provide for future growth because one thinks one cannot afford to do an adequate job is shortsightedness. But to sell water so cheap that one is too poor to have adequate facilities or to provide for future needs is "penny

wise and pound foolish." This is penny-wise water.

In developing the penny-wise theme, let us take a look at the true value of water. Water is the lifeblood of the community. Without an adequate supply of it, no community can prosper and grow. Indeed, if its water supply is not adequate, the community will, as it were, wither on the vine just as a growing plant that is not watered adequately.

In the new bulletin of the American Water Works Association, *What Price Water* (*1*), there appears a statement that vividly describes the true importance of water.

> Water is a commodity so precious that no tyrant has ever dared deny it to his people. The earliest records of our civilization are linked to the spring and the waterhole, the river and the well.
>
> Wars have been fought over water rights and once mighty nations have vanished because their water resources failed. Men have battled to the death over the last few drops in a canteen.
>
> Families have given up their homes and deserted their properties because of failing wells and dried-up water courses. London was virtually destroyed by fire in the seventeenth century and Chicago was reduced to ashes in 1871 because sufficient water could not be delivered to the right place at the right time.

Water is a natural resource and as such must be conserved. It must neither be wasted nor put to use in quantities inconsistent with the supply that is economically available. Just as large electric-power plants are located where coal and large quantities of water are available, so must all industries requiring vast amounts of water be located where water can be readily and economically obtained in the required quantity. Likewise, major centers of population must be located where the water supply is adequate or can be economically developed. Water cannot be economically transported over vast distances, as can oil and gas.

With these thoughts in mind, can we not agree that, if a community is to have a continued healthy existence, it must have an adequate public water supply—whatever the cost?

The Three 'A's'

But what are the attributes of an adequate public water supply? All will agree that water service must be adequate to meet future needs, but some of the factors that help to insure adequate service may not be immediately apparent. In considering what goes to make up adequate water service, I like to break adequacy down into the three "A's": *a* sound system, *a* sufficient income, and *a* public that fully appreciates the value of water service. We must have all three or we cannot do the job.

Now let us review what goes to make up sound systems. To have a sound system, the water supply must be adequate, as must plant facilities. The distribution system likewise must be adequate. To accomplish all of this requires that there be an adequate money supply as well as an adequate water supply. This brings us to our second requirement—sufficient income. To have sufficient income, we must have high-enough rates to pay for necessary expansion of facilities, proper operation, and adequate salaries to personnel. To build hand-to-mouth facilities, to skimp on operation, and to underpay our employees in order not to raise water rates is again penny wise and pound foolish—penny-wise water, again.

But if we are to have adequate rates, our customers must be willing to pay them, and that brings us to the third

"A"—a public that is appreciative. If we are to have adequate appreciation of the value and importance of adequate water service and the necessity for paying adequate rates for water, we must make our customers aware not only of the importance and value of our product, but also of the many things that have to be done to the water before it reaches their faucets. If we are to develop such appreciation, we must at the same time do what it takes to make our service worthy of appreciation. From the standpoint of management as well as operation, we must deserve appreciation.

Anticipated Future Growth

Acting through its Water and Sewerage Industry and Utilities Division, the US Department of Commerce, in January 1956, released a report in which it projected the national water usage from 1955 to 1975 (*2*). This report shows that the use of water by United States public water supplies may be expected to increase 64 per cent during the next 15 years. This anticipated national increase is sufficient to cause those responsible for the administration of public water supply agencies, however small, to make ready for the future by planning necessary plant expansion and charging rates adequate to pay for the expansion required. To do otherwise would be to invite penny-wise water.

Determination of Adequacy

As the first step in determining the adequacy of the community water supply, water utility officials should study operating records and determine how well the water system has met the demand during drought years. From this it may be deduced whether the supply can be expected to meet future drought demands. Future population projections should then be made in order to estimate future water supply requirements. These should be compared with the available supply and with the capacity of the existing plant.

Determination of the future available water supply and necessary plant extensions can be difficult. In fact, unless the utility has the services of an experienced engineering staff, the problems encountered in making proper comparisons and developing necessary plans are so complex that assistance should be obtained through the employment of a consulting engineer. Not to secure competent engineering assistance because of the cost involved is conducive to penny-wise water.

Master Development Plan

In all but the smallest water systems, a master development plan should be prepared. Such a plan should be comprehensive and outline the general nature, capacity, and approximate cost of the works that will have to be constructed to meet present and future community water requirements. It should be expected that such a plan will need to be revised from time to time to meet changing conditions. It will, if kept up to date, provide an adequate guide in the development of construction programs.

Improvements Cost Money

To build the necessary improvements will quite possibly take a lot of money. It will probably be necessary for the water rates to be raised—perhaps as much as 50 per cent or more. Can it be questioned that, if necessary to achieve safety and adequacy, the rates could even be doubled without being prohibitive? Compare water rates with those paid for telephone service or for electricity and gas; water rates are obviously penny wise.

Building water system improvements and extensions does cost money—and a lot of it—but one can be assured that community money will never be better spent than when those dollars are used to insure that the public water supply keeps pace with the growth of the community.

Let us see what it actually costs to have an adequate water supply. Suppose a necessary improvement project does cost $50,000–100,000, $500,000, or $1,000,000. No single individual is going to pay for it. The only thing of real importance to the individual customer is how much he will have to pay for the water service he receives, not what the aggregate cost of the improvements will be.

Let us see what some of the water uses we take for granted actually do cost. A tub bath may cost as much as 3 cents; a shower bath costs even less. What if it does cost $\frac{1}{3}$ cent to flush a toilet? Would any of us go back to outdoor plumbing to save the cost—at even double the cost?

It costs less than 0.1 cent—not even a dime a month—to wash one's face three times a day. Is anyone going to stop washing his face to save 10 cents a month—or even 20 cents a month, if the rate should be doubled? Certainly not.

Then there is the garden out in back. Perhaps it needs a good soaking. Is anyone going to let it burn up because it may cost 15 cents an hour to water it? Would anybody give up the garden even if he had to pay 20–25 cents so that a new pumping station or filter plant could be built to make the supply adequate? I don't think so. Water is the most necessary thing anyone buys, and, whatever the price, it is worth it.

Water Is Too Cheap

How many of us have been guilty of bragging that water is "cheaper than dirt" or that water is the cheapest thing one can buy? I have. But how can we expect our customers willingly to pay more for what we ourselves say is so cheap? Rather, let us brag that water is the most necessary commodity that our customers buy. Let's not have penny-wise water.

Water rates are too low—we know it, so let's face it. John H. Murdoch Jr., then general counsel to the American Water Works Service Company of Philadelphia, a company operating over 150 water utilities in many parts of the country, presented a paper at the St. Louis convention of AWWA in 1956 entitled "75 Years of Too Cheap Water" (*3*)—penny-wise water, if you will.

Water works systems have always failed and are now failing to offer to their customers the water service desired. Water works men excuse their failures on the ground of poverty and yet are publicly proud of the low prices charged. Service deficiencies will continue until this industry becomes ashamed of too cheap water.

From the early days of water systems to the early 1900's, public water supplies could be created and kept alive only by offering competition in price against wells and cisterns, which were generally more highly regarded.

People saw little need for public supplies for cooking and drinking until they realized the danger of using impure water.

Our water works founders, in order to bring in the numbers of customers required to make the enterprise possible, kept the cost of their plants to the bare minimum and fixed their customers' service charges so low as to be attractive to people who really did not particularly

want the rather inferior service being offered. Understandably, the public came to look on water as cheap.

We claim that we deliver water to our customers as they need it. The claim is not well founded. Water is delivered when it is available at the source, and when and as the pumps and distribution systems are not overloaded. When the customers want it most they cannot get it. We are too poor to make adequate provision in source of supply to carry our demands through a recurrence of recent droughts. We are too poor to develop supply facilities which will be needed to meet increasing demands in the near future although we know that long years of effort must go into such developments.

So we carefully explain to the customers and to the public that the type of service which is wanted and which we know how to render cannot be rendered because we cannot afford it.

Let each man remember that public water supply systems are no longer in competition with backyard wells and cisterns and need not cut service or charges in order to get essential business. Let each remember that the public now considers water service essential and wants that service improved, strengthened, and extended. Then let the water works man, having cleared his mind of the old habits of thought based on long forgotten conditions, begin the process of leading the people to a realization that adequate service such as they want, can and will be given whenever they want to pay the new costs rather than traditional rates.

This goal can be reached only by getting away from penny-wise water. If we are to accomplish this, we must convince the public that water service is not valueless, but priceless.

Self-Appreciation Necessary

Before attempting to sell the public on the value of our service to the community, we must realize, ourselves, that the community could not get along without our product, that we are very important persons to the community. We must upgrade ourselves in the process of upgrading our product. We must employ competent personnel and pay them adequately. A high-grade product cannot be produced by second-rate people. We must think "big."

It helps the prestige of water utility service personnel, meter readers, customer servicemen, and others having personal contact with the public to give them neat, distinctive uniforms. The cost need not be too great. In one city, the water department purchases the original uniforms for its employees and shares replacement costs on a 50–50 basis.

Money spent in increasing the prestige of utility personnel will be money well spent. It will help to make penny-wise water a thing of the past.

Cultivate Public Appreciation

Appreciation of the true value of water service can be promoted by making the public conscious of the fact that water is indispensable, that the water system that supplies it to them is rendering a most valuable service, and that penny-wise water is "pound foolish" (damned foolish, if you ask me).

There are many ways in which this can be accomplished at little cost. I will mention but a few of them. At every opportunity—in discussions with friends, before luncheon clubs, on radio time (if radio stations are available), with bill stuffers, in newspaper stories, or by any method you can dream up:

1. Personalize your employees. Give them public credit for the good work

they are doing. If you do not wish to pat yourself on the back, pat your employees' backs and accomplish the same thing.

2. Tell of the convenience of having plenty of safe water at one's fingertips.

3. Explain the difficulty and expense of producing safe water and distributing it to the customers.

4. Justify your rates by showing how the customers' money is spent.

5. Build good public relations on a personal level; polite cashiers and neighborly servicemen can cultivate much good will without an expensive advertising campaign.

6. Win friends and influence the public by practicing good manners and teaching them to your employees.

7. Above all, provide good service when it is necessary to perform irritating work, such as making street cuts, obstructing traffic, or shutting off the water for any purpose. Explain the need for such work by erecting signs and providing adequate advance notice of shutoff.

Many water utility men already do these things. So can you!

When these things have been done, or better yet, concurrently with them, spend money for advertising. It may be thought that publicly owned utilities cannot spend money for advertising, but they can and many do. They can at least prepare attractive and readable annual reports that will get across the message. In fact, the public has a right to expect such reports.

Service trucks can be attractively and distinctively painted. Hydrants can be freshly painted and building and grounds adequately maintained. Educational pamphlets can be distributed, that can be bought from AWWA at modest prices.

AWWA Can Help to Abolish Penny-Wise Water

AWWA can, in fact, help in many ways. Ten years before John Murdoch so forcefully called attention to the tendency to play down the value of water, the AWWA Board of Directors recognized the problem and set out to assist the water supply industry to better its public relations. A professional writer was employed to prepare a public relations manual. It was titled *Silent Service Is Not Enough* (*4*). In December 1954, the manual was published in the JOURNAL.

To illustrate the principles set forth in the manual and to assist water utility management in implementing programs to better public relations, another publication was developed by the AWWA staff. Originally called *Public Relations at Work,* but currently known as *Willing Water,* it now is published monthly and distributed to AWWA members. Unfortunately, too many of our members who are responsible for the maintenance of good public appreciation and the dissemination of water supply information do not properly value the information included in *Willing Water* and do not read it carefully. Too often *Willing Water* is put aside and never read.

Perhaps a new format would help. Consideration now is being given to making *Willing Water* more attractive and readable. The staff will appreciate having suggestions as to how this may be accomplished.

Cartoons, posters, decals, mats of prepared ads, and other public relations guides have been developed by AWWA and are available at a modest cost. More aids of this sort will be prepared under the advancement program.

A picture book, in comic book format, telling the story of water supply and aimed at the junior customers, can be obtained from AWWA at prices varying from 15 cents to 2 cents, depending upon the quantity purchased. Many utilities have used thousands of them. Get some of these comic books and give them to the school children. Many of the parents will read them, too.

Followthrough Is Imperative

To avoid the pinch of penny-wise water, we must follow through. A single step is not enough to reach the goal. We must turn all our steps in that direction—and AWWA is working now toward leading the way. That is the basic purpose of the new Water Works Advancement Program.

Through education, AWWA will try to stimulate water utility management to change its outlook from penny-wise water to "improved water service, through water works systems self-sustained and adequate to meet the growing needs of each community." Then through both its present publications and a new series of public-information aids, it will endeavor to assist management in local campaigns to enlist public support for water service that is at least up to the standards of other utility services.

Beyond these internal efforts, AWWA expects to participate in a program to build public awareness not only of the value of public water supply, but also of the desirability and feasibility of quality water service. This latter program will be carried on through national publicity and advertising campaigns developed in cooperation with the manufacturers of water works equipment and materials.

An End to Penny-Wise Water

The public has a right to expect that the public water supply system will be adequate to provide water when and where needed. To assure that the utility will not be too poor to give such service, rates must be adjusted to supply enough money to carry on necessary improvement programs. To accomplish this goal, the management of each utility must use all of the means at its disposal to inform the public of the value to the community of an adequate water supply and to convince them that whatever has to be spent to make the supply adequate will be money well spent. Only when this has been done, and not before, can there be an end to penny-wise water.

We already have the means for starting such a campaign on the home front. *Let's do it now!*

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Methods of Financing Water Utilities in Michigan

Louis E. Ayres

A paper presented on Sep. 8, 1958, at the Michigan Section Meeting, Grand Rapids, Mich., by Louis E. Ayres, Cons. Engr., Ayres, Lewis, Norris & May, Ann Arbor, Mich.

THIS article discusses the sources of capital and of income, other than commodity rates, required to finance a water utility. This is a subject of particular interest under present-day pressures of suburban development and the handicaps of legal limitations.

The capital required by privately owned water utilities is limited to borrowed money and is obtained through the sale of common and preferred stocks, debentures, and mortgage bonds, or from bank or other short-term loans. Utilities under state water commission control may not plow earnings back into the capital structure. The income of a privately owned utility is limited to approved commodity rates and fire-protection charges.

Publicly owned water utilities, although much of their capital is derived from borrowed money through the sale of general-obligation, special-assessment, and revenue bonds, may secure additional funds from: taxes; surplus, or the profits of past operations; contributed funds, private or governmental; and various other devices such as the front-foot assessment, or capital, connection, and benefit charges.

Background History

In Michigan's early days water supplies were provided and financed by the family. Water was obtained from wells—individual house wells or community wells—or from surface streams and lakes. The early settlers of Detroit, for example, obtained their water supply mostly from the Detroit River. It was supplied from a pump on a wharf and carried:

> . . . in casks and barrels, hauled in carts, and in buckets slung at either end of wooden yokes; . . . and the ordinances of the trustees compelled each citizen to keep on his premises a cask containing a certain amount of water and so arranged with poles for handles that it could be brought into immediate use *in case of fire* (1).

This is an example of the fire-protection service of a century and a quarter ago.

With the growth of communities to more than a few hundred families the need was soon felt for a system of distribution pipes to supply water for domestic use and fire protection. This objective was often accomplished by the formation of a company of promoters who undertook to supply the funds necessary to provide water, pumps, and a distribution system.

Detroit

On Mar. 30, 1827, a private company was given the exclusive right of supplying Detroit with water. "At that time the city contained about 1,500 inhabitants. For the use of water families were charged $10 per annum (*1*)." Although the company "continued to expand the works in the face of increasing pecuniary loss, it failed to furnish a constant supply . . . of pure, clean, and wholesome water." Its charter was, therefore, forfeited and its property was purchased by the city on Jun. 4, 1836, for $20,000, payable by 20-year, 6 per cent bonds. This was one of the early failures of a private water company. Similar experiences were common in the history of municipal water utilities elsewhere in Michigan.

With the purchase of the company's property Detroit was faced with an expenditure of more than $180,000 for new facilities. In spite of all efforts to improve conditions, "the Common Council was besieged with petitions for relief and for reduction of rates and found itself in nearly the same situation that its predecessor, the Old Hydraulic Company, had been in only 15 years previously." This condition appears to have been caused by the rapid increase in population, inadequate rates, lack of experience in water utility practice, and Common Council management: "Many discussions and debates were held as to the policy to be pursued. . . . Many were in favor of a sale to a private company, . . . others were strongly opposed to such a course. Finally, in February 1852, the council passed an ordinance by which the control and management of the water works were vested in a board of trustees, consisting of five members, and on May 16, 1853, the Board of Water Commissioners was organized." At this time the population of the city was more than 35,000.

This new Board of Water Commissioners, which was created by an act of the Michigan legislature, had broad powers. Pertinent to this article, it had the power to borrow money and to issue bonds, pledging the faith and credit of Detroit for the payment of their principal and interest. The board also had the power to assess a water rate which would: "become a continuing lien" upon any: "house or other building and upon the lot or lots upon which such house or other building is situated." In addition, if any sum was needed: "over and above this revenue . . . to meet the payment of interest or principal of bonds issued . . . it shall be the duty of the Common Council to raise said amount by a special tax in the same manner as general taxes." In 1887, for example, $75,000 was received by the board "from a liquor tax."

From 1876 through 1919, inclusive, more than $3,000,000 was collected by special taxes and paid to the board. In the early days receipts from taxes sometimes amounted to as much as 25 per cent of gross receipts from water assessments. Toward the end of the period money received from taxes dropped to as little as 3½ per cent of gross receipts from rates.

Revenue that now comes from meter sales at first was collected in Detroit by means of flat-rate assessments, established in the bylaws of the Board of Water Commissioners Feb. 14, 1854. The first meters appeared in 1878. By 1890, 10 per cent of water services

were metered, by 1924, 50 per cent, and by 1930 almost all sales were metered.

Types of Charges

1. *Front-foot assessments and charges.* The front-foot benefit charge is an annual charge and a contribution to income. It should be distinguished from the commonly used front-foot assessment which is levied on property at the time a main is laid and is intended to cover, in part, the capital cost of the main extension. Front-foot assessments are common but are generally inadequate to cover the cost of the mains laid.

In a report to Detroit for the year ending Dec. 31, 1869 (*2*) it is stated that: "The legislature of this state, at its last session, passed an act making it the duty of this board to assess an *annual tax of 3 cents* per lineal foot of the frontage of lots in front of which water pipes are laid and which do not pay water rates." This tax, first assessed for the year 1870, amounted in that year to $6,446.40, about 4.65 per cent of total receipts from water rates. "The assessments were made annually, but no proceedings were had to enforce collection by sale of delinquent premises," until the beginning of the year 1876: "Upon suit brought by parties interested the Supreme Court of the state, at its last June [1876] term, declared the act in question invalid. All amounts collected under the act were repaid (*3*)."

It is interesting to note that in 1918, 42 years after the Michigan act, assessing an annual tax on lot frontage of nonusers was declared invalid. Maryland passed the enabling statute of the Washington Suburban Sanitary District, which provided that: "debt service on district bonds for capital cost be met by a low tax on all property in the district, plus a front-foot benefit charge on properties abutting water and sewer lines (*4*)." This benefit charge has now been collected for 40 years and in 1955 it accounted for about 31 per cent of the total income of the district.

Apparently an annual front-foot benefit charge is not only unconstitutional in Michigan but: "Legal authority is wanting in many jurisdictions (*4*)." Courts find that it violates a basic rule of taxation which requires uniformity of application of benefits.

2. *Capital charges.* Capital charges are collected either as a lump sum to aid in plant expansion or as an annual addition to rates, equivalent to an extra demand or service charge.

3. *Connection charges.* Connection charges are made to reimburse the system for the cost of services and of meters installed.

4. *Customer-benefit charges.* A customer-benefit charge permits new customers to buy into an existing water system. Such benefit charges, for both water and sewage service, were defined and adopted by an ordinance of the Common Council of the city of Ann Arbor, Mich., Jan. 16, 1956:

WHEREAS, there are certain properties in the city of Ann Arbor which have not been developed to the extent of being connected with the water and . . . sewer systems owned and operated by the city, and

WHEREAS, a benefit charge has accrued from said properties by reason of the construction and maintenance [of these systems] which afford and make available to such properties water and . . . sewer service since the inception of these services by the city, and

WHEREAS, such properties have not contributed any reasonable or appreciable amount toward the cost of construction

of the existing [systems], and by reason thereof a benefit charge against such property should be made before such property is permitted to connect to the water and . . . sewer systems of the city to cover the property's fair share of the cost of such existing services.

Now, therefore, be it resolved, that a benefit charge shall be due and payable at the time of the installation of the water meter for . . . properties . . . not heretofore connected to the water and sewer systems of the city.

The schedule of benefit charges adopted by this ordinance varies from $75 for a single-family residence, $120 for two-family residences, $200 for four-family residences and up to $2,000 for 100-family residences, plus $20 for each family in excess of 100. There is also a meter schedule covering industrial, commercial, and miscellaneous, buildings which ranges from $75 for a ¾-in. meter to $6,800 for a 6-in. meter. A later amendment provided that those properties that were annexed to or a part of the city at the time of passage of the ordinance would be granted a 5 per cent discount per year for every year they were part of or annexed to the city prior to 1955. This discount may not exceed 50 per cent.

It seems to the author that such a so-called lump-sum benefit charge, which requires a later user to purchase his proper share in a plant that is operating and paid for by earlier users, offers a partial alternative to the Washington Suburban Sanitary District's annual front-foot benefit charge. This charge also is intended to collect in advance a proper share of the investment, to be credited to the later user when he subscribes to the water system. It is not practical, however, to make the lump-sum charge nearly as effective as the annual charge. The Ann Arbor lump-sum charge of $75 per single-family dwelling may be compared with the annual charge in Maryland on a 60-ft lot of $12 per year for 40 years, the life of the water district's bonds.

5. *Fire-protection charges.* Years ago, it was a generally accepted theory that the income required to support a water utility should be derived from two sources: sale of a commodity and payments for services. The commodity was purchased by the individual customer and paid for through water rates; the services were an obligation of the community as a whole and were paid for through general taxes.

One service in particular that has been accepted since the first days of water utilities is fire protection. In fact, in many instances, it was the prime function of the utility. It was the one public service that appeared to be susceptible of evaluation in dollars and yet it has been the subject of controversy for 50 years. Other services to the community, accepted as real and important yet not subject to precise evaluation, are those related to public health, sanitation, and the general well-being of the public. In recent years payments for fire protection have lapsed largely, except where maintained by state regulatory commissions. Although all privately owned water utilities are under commission control, municipally owned utilities are usually free from such supervision.

6. *Other charges.* Expediency in taxation brought about the abandonment of a charge for fire protection because it involved a payment to the water utility that had to be appropriated by the city budget, which is burdened by many other demands for funds. However, special assessments

to provide distribution mains and extend mains, capital charges to enlarge plants, connection charges to help pay for meters and services, and benefit charges to permit new users to buy into existing systems are all essentially charges to property, but not to the same property covered by the general city tax. All of these devices are available for use by the publicly owned water utility.

Extreme bases for financing a water utility would be to make "use of general tax funds to finance . . . major structures," the "use of special assessments to finance . . . local structures," or the "use of water charges to finance operation and maintenance (*4*)." However, the propriety of reducing water rates so that they cover only operation and maintenance charges under normal conditions may also be doubted. To do so might reduce rates unduly and lead to waste. The objectives in rapidly growing areas are to: secure the revenue necessary to finance water-service additions without placing too much extra financial burden on existing users and to finance service to new developments without resorting to excessive rates.

Legislation

Much legislation has been passed in Michigan in the last 30 years in an effort to aid in the financing of water-supply facilities in rapidly growing areas. In general, the purpose of the legislation has been: "to provide the necessary powers to larger governmental units, such as metropolitan districts, counties, and authorities to own and operate the required facilities and to finance the costs with the aid of both taxes and rates (*5*)."

The first act to create a metropolitan district was Act 312 in 1929:

This act authorized the formation of a combination of two or more cities, villages, or townships to own and operate a water system, with only a limited power to tax in a special assessment district but with authority to sell mortgage and (later) revenue bonds. This metropolitan district idea was promulgated by the city of Detroit, with the expectation that an agreement could be reached on the organization of a water district to include Detroit and the surrounding areas. The plan never materialized (*5*).

The Revenue Bond Act of 1933 was the next important step in the financing of water utilities and under its provisions many projects have been financed. It has served well in the past in connection with extensions and improvements to existing municipal plants. It is, however, inadequate to finance improvements for undeveloped areas if there are not sufficient customers at the beginning of a project to provide the necessary revenue without prohibitive water rates. One reason for this is that in financing with revenue bonds it is necessary to provide enough revenue to pay all operation and maintenance expenses plus 150 per cent of the debt service.

In 1939 the so-called County Plan was authorized by Act 342:

> . . . to fill the gap in the Detroit area left by the previous failure to form a metropolitan district. Under this plan the Board of Supervisors can authorize the formation of a water district within the county and finance the same by the sale of general-obligation or revenue bonds, and may tax the property in all the cities, villages, or townships served, and charge rates to the users of water. Under this law the Wayne County Road Commission developed a system of water mains through which the western portion of Wayne County is supplied with Detroit city water (*5*).

In 1952 Act 196 authorized:

> . . . the formation of water authorities by the incorporation of any two or more cities, villages, or townships, with power to own and operate a water system, sell revenue bonds, and contract with its constituent units for the sale of its product. These bonds were made good by contracts with each city, village, or township which were severally authorized to raise by taxes and by rates the sums necessary to meet their respective obligations (*5*).

The cities of Berkeley, Birmingham, Clawson, Huntington Woods, Pleasant Ridge, and the township of Southfield were thus: "organized into an authority to purchase a water supply from Detroit and distribute it through master meters to the boundaries of the several communities (*5*)." This authority now includes the cities of Southfield and Lathrup and the village of Westwood.

In spite of the legislation of the last 30 years, and particularly of the last 5 years, the financing of water supplies in rapidly growing and extensive suburban areas presents problems for engineers, attorneys, and financial advisers. Additional sources of revenue, other than from water rates, have been sought. For example, in 1955 Act 233 authorized the use of state tax money and: "any other funds which may be validly used." And Act 4, 1957, authorized financing by general-obligation bonds and stated that a: "tax for the purpose of paying the bonded indebtedness shall be unlimited as to rate or amount."

Difficulties in financing will continue so long as dependence is placed on the use of general taxes collected from an entire township or other political subdivision to supplement the income from rates collected from only a portion of the area involved. Rates are paid by users; taxes may be collected largely from nonusers of the service. Equity requires that both the users and the nonusers—or, users of the future—support the project. Although front-foot assessments and lump-sum benefit charges supplement rates no better device has been adopted than the annual front-foot benefit charge of the Washington Suburban Sanitary District, previously described.

Southeastern Oakland Authority

The Southeastern Oakland County (Mich.) Water Authority presents unique and outstanding examples of both water-distribution methods and the allocation of costs to customers. Water purchased by the authority is limited to a rate of demand not to exceed the average rate of maximum-day use and it is delivered to its constituent units at the demand rates required to meet their respective maximum hourly use. All water received and sold is measured through recording meters. The records of the authority, therefore, provide unusually complete data on hourly rates of water use, storage requirements, and related factors of importance in the intelligent design and operation of water systems.

This authority is also unique in its plan for distributing costs. In the original contract each unit purchased an estimated maximum daily demand for the year 1970 and each agreed to pay as a demand charge its proportionate share of 50 per cent of the total fixed charges. The balance of the fixed charges, plus operating costs, are included in a uniform water rate. Also, as time goes on, each unit will have the right to buy additional water or sell unneeded water at the originally established purchase price, cor-

rected for compound interest to the date of purchase or sale.

The Southeastern Oakland County Authority will have increased its estimated water demand in 10 years (1950–60) to the growth anticipated for 20 years (1950–70). The contract has provided for this increase. The arrangement is generally accepted as successful and equitable. It affords an alternative method for the financing and cost allocation of regional water supplies elsewhere.

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Initiating a Safety Program in the Philadelphia Water Department

Victor A. Appleyard

A paper presented on Jun. 25, 1958, at the Pennsylvania Section Meeting, Erie, Pa., by Victor A. Appleyard, Chief of Water Operations, Water Dept., Philadelphia, Pa.

MANY articles on water department safety and a safety manual (*1*) have been published by AWWA. The National Safety Council, Chicago, has also published a great deal of information on this subject. However, most of the published information is in general terms and covers fundamentals only. When it comes to translating the data into a planned program for a specific organization the water official must become truly realistic and practical.

What works for one organization may not for another. This stems from the fact that every organization has a personality, made up of the composite thinking of a number of its executives and all of its supervisory personnel. The policies and working organization stem from these men and that is why all water utilities are different, a fact that must be recognized when starting a safety program. Therefore, setting up a program is not a simple assignment, especially in a medium-sized water company or department.

Background

In 1952 Philadelphia started operating under a new city charter which combined the water and sewer utilities into one department. The revenue received for these services can, under this charter, only be used to operate the department and the department's commissioner has the responsibility of operating it as a self-sustaining unit. Revenue must equal expenses. This reorganization started Philadelphia's water department off on a sound basis.

In 1953 all employees of the department were reclassified—a tremendous task. By this time the department had launched a large rehabilitation and new construction program, costing about $30,000,000 per year.

In spite of the inevitable confusion during the period 1952–53 the safety of the employees was not forgotten by the executives of the department. An informal safety program was in effect throughout the department, emanating from the commissioner and implemented by the supervisory staff. One division established some procedures and a formal program.

The department has four major divisions: water operations, with 1,040 employees; sewer operations, with 265; design and construction, with 340; and administration, with 55. Altogether the department has a total of about 1,700 employees. The safety engineer functions in the administration division, which is under the authority of the commissioner's office.

Beginning of Program

In November 1955 the commissioner created a five-member departmental safety committee under the chairmanship of the author. All four departmental divisions were represented on this committee. The committee's assignments were to: reorganize the safety and accident prevention program; standardize safety practices and procedures and recommend new ones; work with the National Safety Council and the local safety council in order to introduce current safety practices; and set up safety subcommittees within the department's units. A further responsibility of the committee was to review all accident and safety reports, develop statistics from them, and utilize this information to reduce the accident rate.

The departmental safety committee met fairly regularly but too much time elapsed between meetings. Most of the committee members were line supervisors, and the departmental reorganization going on left little time to devote to the details of setting up an adequate safety program. The committee decided that the best approach to the whole problem was to find out what the department's safety record was, particularly in the various operating units. A survey of accident reports, however, showed that:

1. Many reports were not complete
2. There were often serious delays in processing reports
3. Not all accidents were reported
4. Many reports were not followed up and no statistics were compiled
5. No differentiation was made between chargeable and unchargeable accidents.

In other words, the department had been "preaching" safety but doing little to achieve it. The committee's conclusion was that a full-time employee was needed to head the program. A safety engineer job classification was created, and a man was hired as department safety engineer in February 1957.

In 1956 the department had a thorough review made of the motor vehicle accidents for 1955. This area seemed to need the most immediate attention. The accident records available were excellent, probably because the State Motor Vehicle Bureau requires accident reports within 24 hr if personal injury or $100 in property damage is incurred. One action that was taken by the department as a result of this review was to require a physical examination—paying particular attention to the eyes—of its drivers who had had two accidents within 12 months. So far this action has been quite effective; to supplement it the department is now instituting a driver test by the state police to cover vision, distance judgment, steering, brake reaction, and field of vision.

Since the safety engineer was employed, the committee has implemented the program with much more vigor. Records have now been set up in a system and monthly statistics which can be relied upon are available. These statistics were put to use when the safety engineer discovered that the department's ratio of lost-time to no-lost-time injuries was grossly out of alignment compared to national statistics. This was investigated and it was learned:

1. Philadelphia's clinics and doctors were extremely liberal in permitting an injured employee a day or two off for a very minor injury.
2. The method of processing an injured employee at the clinic permitted him to take time off and the supervisor could not challenge him without

spending an excessive amount of administrative time on the problem.

Corrective action has been taken and there has been a considerable improvement in the department's record.

Safety Subcommittees

Accurate records and expeditious handling of accident cases do not *prevent* accidents. A third step was therefore instituted by the committee: the formation of safety subcommittees in all operating units. A review was made of the safety committees already in existence in the department's three sewage treatment plants. They consisted of three men—workmen—who made a plant inspection about once a month and reported to the supervisor in charge of all the plants. The departmental safety committee, after due consideration, felt that similar subcommittees should be set up in the water operations division. There are six major sections in the water operations division: pumping, distribution, filtration, meters, plant maintenance and garage, and customer service.

The committee directed the safety engineer to interview the section supervisors after orders had been issued for the setting up of safety subcommittees. As could be expected he encountered varying degrees of cooperation, ranging from the prompt appointment of a three-man committee to "It can't be done by my shop." This latter attitude was resolved by having a higher-level supervisor not only support the program but also meet the men in that shop, with the safety engineer, discuss the problem and resolve it. It should be mentioned that the department's management is completely in favor of the safety program—an essential requirement for its success.

The subcommittees generally consist of three men. A foreman can be a member but workmen are usually preferred in order to keep it at a "grass-roots" level. Each subcommittee has a chairman, a vice-chairman, and a secretary. Each member serves 2 months in each capacity. The terms are staggered so that every 2 months the chairman leaves the committee, a new member is appointed secretary, and the other two members become chairman and vice-chairman. This procedure guarantees continuity and gives everyone a chance at the coveted position of chairman. With the right guidance the members will continue to be safety exponents long after serving on the committee.

Each month an inspection is made of the plant or plants by the subcommittees. The members are given a check-off sheet—with room for comments—to guide them on points it is especially important to inspect. Their report is submitted to the supervisor of their respective area, with a copy to the safety engineer. They take part in a safety meeting called by the supervisor and their recommendations are discussed. The safety engineer follows up the reports with the supervisor if requested to do so or if a particular recommendation is repeated in two successive reports, indicating that no action has been taken by the supervisor. If a policy decision must be made it is referred to the department safety committee.

There are sixteen subcommittees in the water operations division: two in pumping, four in filtration, two in meters, two in distribution, three in sewer treatment plants, one in sewer maintenance, one in plant maintenance and garage, and one subcommittee in construction.

Results

From its beginning the safety program has had a good general reception. The departmental safety committee is convinced that the department's safety record will reflect the efforts being made. The committee calculates that this organization in safety will cost the department about 30 man-days per month, or only 0.08 per cent of total man-days worked in the department. According to the department's lost-time record for the first 5 months of 1958 347 man-days were lost as compared to 747 man-days lost in a similar period in 1957. Disabling accidents were reduced from 49 to 37 for those same periods. To those managers who want a "payoff" on a program, here it is. An additional factor is the improved employee morale. It can easily be agreed that working the safe way is also one of the most efficient ways.

Even previous to the formal beginning of the safety program steps were taken which showed that management and some of the employees were aware of the problem. Safety material, such as hard hats for ditch gangs and first-aid kits in all plants and vehicles, was distributed in a number of areas. Traffic cones and barricades were furnished for street-working teams. Inspections by the Philadelphia Fire Department led to the installation of fire-fighting equipment. Working clothes, goggles, face shields, and gloves were furnished by the Water Department in various working areas.

Many of the executives and employees are now thinking seriously about safety practices. One day a supervisor called the author to request a change in the specifications for an order of rain gear. The order was about to go out for bids but the author had it held up long enough for the supervisor to change the specified color from the usual black to bright yellow. This was a heartening sign of an increasingly thoughtful attitude toward safety.

The committee has asked the department's safety engineer to write up safety standards, first a general one for any and all areas and then specific ones for individual divisions. This is a big job and will take time. In the meantime the department is using the National Safety Council poster service extensively and is distributing magazines and bulletins. The department's monthly house organ publishes statistics and general safety news.

It was decided by the committee to enter the National Safety Council's "Safe Driver Award" campaign. In 1958, 143 awards will be made to employees in the department who drive regularly and did not have an accident in the year 1957. A safety program must have targets to hold continuing and active interest, and when the targets are attained recognition must follow promptly.

Summary

To summarize, the following points should be accented in planning a safety program:

1. Top management must be "sold" on safety.

2. A safety committee for the entire department should be set up.

3. For the larger utilities a safety engineer should be employed on a full-time basis.

4. Accurate records of accidents, injuries, and other factors should be kept.

5. Accident cases should be handled expeditiously.

6. Safety subcommittees should be set up in the divisions and sections of the department.

7. Regular and thorough plant inspections should be made.

8. A promotion campaign to publicize safety to the employees should be started.

9. Adequate safety equipment should be furnished.

10. Awards should be made to safe drivers and working individuals or groups when merited.

One fact will demonstrate the effectiveness of the safety campaign in the Philadelphia Water Department: in 1956 the department's accident frequency rate was 39.08 lost-time injuries per 1,000,000 man-hr; in 1957 the rate had been reduced to 26.41. On the basis of this 32 per cent decrease in accidents in 1 year the Philadelphia Water Department has qualified for the AWWA "Award of Progress." In time the department hopes to qualify for the AWWA "Award of Honor" for a frequency rate of less than 10.

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Laboratory Culture of Taste- and Odor-Producing Aquatic Actinomycetes

J. K. G. Silvey and A. W. Roach

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THERE are many opinions concerning the factors contributing to and causative organisms of tastes and odors in water supplies. The authors believe that the organisms largely responsible for the production of earthy, musty, and woody odors are the aquatic actinomycetes (*1*). A recent paper by Sigworth (*2*) indicated that according to questionnaires most people attributed tastes and odors to algae, decaying vegetation, industrial wastes, and other causes, in the order named. There was some indication that a number of people were interested in the aquatic actinomycetes and their production of tastes and odors. As capably explained in that paper, the techniques employed for the isolation and identification of actinomycetes may offer many distressing problems. Algae are sufficiently large to be observed, and because they have been continuously associated with occurrences of tastes and odors, it is readily understandable that their mere presence might be considered sufficient evidence for the assumption that they are the cause.

The relationship between the actinomycetes and various types of algae has been investigated both in the laboratory and field (*3*). By various techniques it has been found that the aquatic forms can be isolated from either water, mud, or algae (*4, 5*). Laboratory techniques have been devised that provide information concerning the life-history and morphology of the actinomycetes. Information is now available concerning the conditions under which the actinomycetes may grow, the stages they assume, and some of the varieties of taste and odor associated with these stages in natural waters (*6, 7*).

The purpose of this paper is to describe some techniques and equipment designed for rearing the aquatic actinomycetes and the methods for the concentration and study of the taste and odor compounds. Some of the larger laboratories associated with water purification plants or state and city health departments may be interested in investigating some of the techniques described. The results obtained are encouraging and make it possible for the taste and odor compounds to be studied during periods when they are not present in the raw water supplies. The techniques offer adequate opportunities for study of the biochemistry of these organisms and for chemical studies on the taste and odor compounds they produce.

Life History and Morphology

In order to familiarize the reader with the conditions that are necessary for the growth of the aquatic actinomycetes, some general remarks should be made concerning their life history.

The aquatic forms are primarily aquatic because of the requirements of the primary stages of the actinomycetes. Spores arise from the secondary stages and germinate in the aquatic environment, giving rise to minute, ball-like structures whose morphology and characteristics are different from those of the larger secondary stages (*7*). In an aquatic environment the primary stages may form in the fluid medium at various depths, depending upon nutrients, the concentration of oxygen, and thermal conditions. The spores will not germinate without oxygen. This is why lakes in a state of thermal stratification may occasionally have actinomycete tastes and odors. At the termination of thermal stratification with either the spring or fall turnover, depending upon the geographic location, the lake may become homothermous for a period of time and contain dissolved oxygen from the surface to the bottom. In such instances the primary stages grow throughout the body of water. This applies to natural lakes, rivers, and to artificial reservoirs. Temperatures below 7°C inhibit development of all stages. The optimum for most species appears to be around 25°C although they will grow at lower temperatures, particularly in the Middle West.

In addition to moderate temperature and oxygen, organic carbon and some form of nitrogen are necessary for the development of the primary stages. If these stages become exposed to the air, drying effects reduce the viability of these forms, and growth ceases. The extent of development that the colony attains under a given set of conditions is dependent upon the species or variety under investigation. Most of the species found in the United States produce primary-stage colonies that measure less than 1 mm in diameter. They are rather diffuse in form and vary in color from bright yellow to brown, depending on the variety. Collection of the primary stages from natural waters is difficult because fragmentation occurs and the filaments composing the colonies are so small that even microscopic observations are difficult. Moreover, the transparency of the filaments duplicates that of the water, and staining techniques must be used in order to distinguish the organisms.

The duration of the primary stage is normally very brief. This, of course, is dependent upon the general conditions of nutrition, the organisms associated with the actinomycetes, and the temperature of the water. The primary stages during their development may produce organic compounds that diffuse into the water and contribute to tastes and odors. They may occur concomitantly with larger organisms to which the taste and odor compounds produced by the primary stages are frequently attributed. The typical odor compounds produced by the primary stages are fishy, grassy, hay-like, and potato bin. At the conclusion of the primary stages, the odors may either be reduced or completely altered. The primary stages give rise to intermediate gametes, or sexual stages, that unite to form motile secondary stages. These are produced in spring, summer, or fall in most water supplies—whenever conditions throughout the lake, river, or reservoir are conducive to

their development. In shallow reservoirs, or rivers that are consistently well-mixed, these organisms may grow from early spring through late fall.

The secondary stages of the aquatic actinomycetes are larger and their colony development is more diverse. The filaments comprising a colony are large enough to be readily discerned in a microscopic field. Because their transparency is about the same as that of water, they are not readily observed unless they have been properly stained. Secondary stages may either exist in a subaquatic environment or become emergent, depending upon the variety (*8*). Those forms that grow submerged give rise to musty, earthy, and woody odors that commonly occur at the same time or slightly after diatoms or blue-green algae bloom. The processes of the biodynamic cycle that give rise to the aquatic actinomycetes are similar from a nutritional standpoint to those that give rise to algae. The secondary stages of the aquatic actinomycetes often grow best on the margin of a lake, stream, or reservoir. Muddy shoals with white or grey growths near the waterline have been observed by most water utility personnel. More detailed observations reveal that the organisms also grow attached to the mud, sand, and algal mats beneath the water surface. In reservoirs where prevailing winds cause a shift in water level the actinomycete growth may become so dense as to form a continuous mat that from time to time is covered by the receding water. These areas exhibit intense odors.

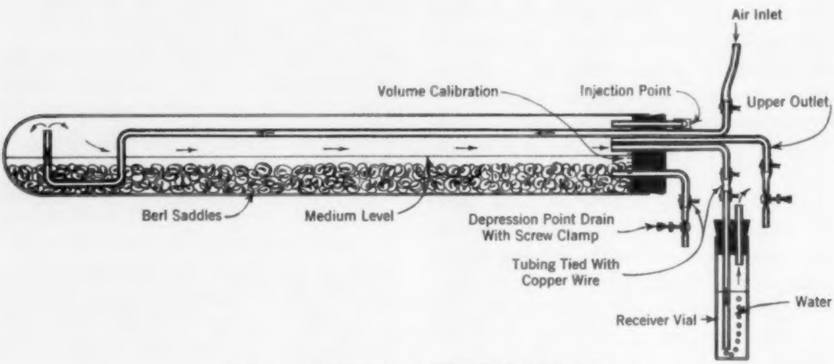

Fig. 1. Diagram of the Nutrition Tube

This is the nutrition tube as set up for the experiments, except for the receiver vial, which was not usually attached. The receiver vial was used to collect any byproducts of the actinomycetes that might have been released into the air stream.

The secondary stages of the aquatic actinomycetes normally give rise to two types of spores, both growing within the same colony. One group of spores will form a cluster, composed of two, three, or four spores. The other variety, produced either at a later time or simultaneously, occurs in chains. Because the aquatic forms produce both types of spores simultaneously or in alternating sequences,

it is apparent that they differ biologically and morphologically from the actinomycetes described from other environments.

There are numerous methods by which actinomycetes may be investigated in the laboratory. They may be reared in test tubes on a solid or liquid medium, in petri dishes on solid medium, or in flasks or larger containers on various types of media. It is readily understandable that the normal, routine laboratory procedures could not be used in a comprehensive study of the nutritional requirements of these actinomycetes. A study of their nutrition that did not include the production of taste and odor compounds would hardly reveal all of the ramifications of their life history. Because these organisms are important from the standpoint of taste and odor but not for antibiotic production, equipment had to be designed that would make it possible to study several factors simultaneously.

Nutrition

The culture containers provided for nutrition studies were test tubes, 50 × 6 cm. The total assembly is illustrated in Fig. 1, which shows the air inlet, air outlet, and injection tube for the introduction of spore suspensions. Sterile air in measured volumes was passed through flowmeters to each tube so that volumetric determinations (in liters per hour) could be made of the quantities of odor-producing materials liberated from the tubes. Batteries of twelve culture tubes were assembled so that a single species of the actinomycetes could be studied with various available nutrients. Figure 1 illustrates the nutrition tube as it was equipped for the studies on liquid medium. A known quantity of large ceramic (berl) saddles was added to each of the culture tubes at the time of dry sterilization to provide a substrate for the growth of the actinomycete mat and so that gravimetric determinations could be made of the amount of actinomycetic mat. In the routine use of the nutrition tube it was assembled as shown in Fig. 1 except that the receiving vial was not attached. The entire assembly was autoclaved in a dry state, after which sterile nutrient was added and reautoclaved. After it had cooled in the sterilizer, each tube was removed and mounted on the plexiframe and attached to the sterile air system. The injection point was comprised of a glass tube closed on the outer end by a rubber vacuum cap so that spore suspensions could be injected into the nutrition chamber. The air inlet admitted calibrated quantities of air continuously and any actinomycete byproducts in the air stream were scrubbed out in the receiving vial. Each vial was immersed in a small Dewar flask and maintained at a temperature of 0°C in order to increase the solubility of the escaping vapors in the water. Wherever it was desirable to retain all materials in the air stream the vials were partially submerged in a Dewar flask containing a mixture of acetone and solid carbon dioxide so that the odor compounds were frozen. This provided a means for calculation of total organic materials produced with a particular nutrient by either the primary or secondary stages of a known species of aquatic actinomycete. The upper outlet could be used when the necessity arose to bypass air because no growth occurred or because it was desirable to bring about rapid evaporation of the

liquid medium. The depression point drain was provided so that liquid medium could be removed at any time and subjected to analysis in order to determine the amount of unused material. Figure 1 shows the volume markers of each tube, which were calibrations in the region of the rubber stopper. The calibration showed the volume at the time the tube was first put in service, as well as the residual volume, so that analytical determinations on the medium remaining could be brought into equilibrium properly.

Solid medium was employed in the initial studies on actinomycete nutrition. The primary investigation was of the effects of inorganic compounds on various species of aquatic actinomycetes. In the study of nutrition requirements on solid medium, some type of measurement was necessary to evaluate the growth of the secondary mycelium, or mat. One species of aquatic actinomycetes was used in each sequence of 12 tubes introduced as a measured spore suspension. An illuminating device was made that would

TABLE 1

Efficiency of Nutrients for Waksamara brevipyrosporulata *Cultures*

Test Period *days*	Odor Type†	Potassium *ppm*	Chlorine *ppm*	Ammonium *ppm*	Nitrate *mg per cent*‡	Sucrose *per cent*	Chemical Oxygen Demand *ppm*	Mycelium (dry wt.) *g*
Tube 1*								
5	F	960	922	815	281	1.6	1.2	—
10	G, Ma	950	920	720	274	—	2.4	—
15	Ma, Ms	942	920	640	272	1.2	3.6	—
20	W	930	920	560	266	—	4.4	—
25	W, E	930	916	540	264	0.4	4.8	—
Tube 2*								
5	F	952	920	810	281	3.2	1.6	—
10	Ma	940	918	710	273	—	2.8	—
15	Ms†	932	918	630	270	2.8	4.2	—
20	W, E†	921	916	540	264	—	5.8	—
25	E†	916	914	520	363	1.2	7.6	12.6
Tube 3*								
5	F, Ma	942	918	800	280	6.4	1.8	—
10	Ma†	920	916	700	270	—	3.9	—
15	W, E†	911	916	600	268	4.6	5.9	—
20	E†	906	912	522	260	—	8.7	—
25	E†	902	908	504	260	2.0	11.4	38.4
Tube 4*								
5	F, Ma	950	918	810	280	9.6	1.6	—
10	Ma	940	918	710	270	—	3.0	—
15	W†	936	916	620	268	7.0	4.4	—
20	W, E†	930	912	532	266	—	6.2	—
25	E†	918	912	516	260	5.6	8.8	28.0

* Medium was 0.2 per cent KCl, 0.4 per cent NH_4NO_3, plus tap water; sucrose contents for Tubes 1–4, respectively, were 2, 4, 8, and 12 per cent.
† Key: G, grassy; F, fishy; Ma, marshy; Ms, musty; W, woody; E, earthy. Within the column, dagger indicates a strong odor.
‡ Milligrams per 100 g.

fit securely onto the top of each of the horizontal nutritional tubes. The instrument projected a beam of light that was picked up on the opposite side and was so calibrated as to read in terms of absorption or transmission of the light beam by the agar-agar medium. Various colored light sources were tried in order to determine minimum absorption, although none was superior to the light characteristics produced by an ordinary tungsten bulb. To determine the effect of any nutrient on colony development, readings were made of light absorption through two tubes containing a medium of 3 per cent agar-agar in water; these were controls in this study. Because all nutrition tubes contained a medium of similar thickness and none of the nutrients affected the absorption of light, it was assumed that an increase in absorption was caused by more prolific growth of secondary mycelia as a response to specific nutrients.

The inorganic nutrients investigated in the first set of nutrition tubes were chlorides of potassium, magnesium, sodium, calcium, aluminum, and iron. The only cation that appeared to stimulate mycelial development was potassium, although it was obvious that chlorides alone gave an inadequate anion source for good growth. In continued studies, the same cations were included, supplemented by ammonium and varieties of anions, such as sulfates, bicarbonates, phosphates, nitrates, and nitrites. In all, fifteen sets of twelve nutritional tubes were studied over a period of about 2 years. It was found that sodium, magnesium, calcium, iron, and aluminum were tolerated by, although not specifically required for, development of secondary stages on the 3 per cent agar-agar. Potassium stimulated growth in all species of actinomycetes investigated. The concentration producing optimum growth appeared to be considerably higher than would be encountered in nature. It is possible that supplements of organic nutrients might reduce the quantity of required potassium. Of the anions it was found that nitrates were readily metabolized, whereas sulfates, chlorides, and bicarbonates were important in affecting the hydrogen-ion concentration, but otherwise ineffective. The presence of orthophosphates in all media was necessary for growth. The quantities required were 0.02–0.14 ppm, depending on the species. Phosphates, sulfates, chlorides, and bicarbonates were included in the basic medium in all instances in quantities necessary to adjust the pH to 6.8–7.2. It was concluded from these nutrition investigations that the aquatic forms demanded some type of organic carbon; as long as agar-agar is available, it appears to be adequate to supply the quantities desired. Some form of nitrogen is necessary for the growth of actinomycetes. It appears that either nitrate or ammonium may serve to bring about growth, although ammonium is definitely superior. Organic nitrogen compounds were not investigated, but they are being considered in current studies, and it is obvious that they are much more efficient.

Nutrition investigations employing liquid medium were initiated soon after the nutrition tubes were shown to be efficient for metabolic studies. The medium was composed of potassium chloride, ammonium nitrate, sucrose, and tap water containing 255 ppm sodium, 155 ppm sulfates, 210 ppm bicarbonates, and 0.2 ppm phosphates. One variable constituent was included in each battery of four tubes. For example, when potassium chloride and

ammonium nitrate were in constant quantities, the amount of sucrose was varied from 20 to 100 g/l (Table 1). The nutritional tubes in each instance were injected with the same species of actinomycete. Chemical studies were done on samples of the medium that were collected from the tubes every 5 days for a period of 25 days. The potassium residuals were determined on a flame photometer *; chlorides were titrated; ammonium was determined by nesslerization; and the amount of sucrose evaluated by use of a refractometer. Studies on the exhausted volatile organic byproducts produced during development of the actinomycetes included chemical oxygen demand and quantity of materials accumulated. These data are incomplete, but they are of value because they show the concentration of odor compounds obtainable by this technique. The results show that concentrations of 0.8 ppm odor compounds may be accumulated in water in a period of 24 hr at room temperature. Immersion of the receiver vial in ice water for the same period will yield 5 ppm, whereas immersion of the vial in carbon dioxide and acetone produced a sample containing 200 ppm odor compounds. At the termination of a 25-day period of growth, the mats of the actinomycetes were removed from the tubes, disassociated from the berl saddles, and weighed (Table 1). In these investigations the efficiency of the nutrients was judged on the basis of the total weight of the mat produced in 25 days. Optimum quantities of each nutrient were evaluated on the same basis. It is interesting to note that the ingredients that stimulated growth on the agar-agar were effective in stimulating growth in a liquid medium. Current investigations indicate that

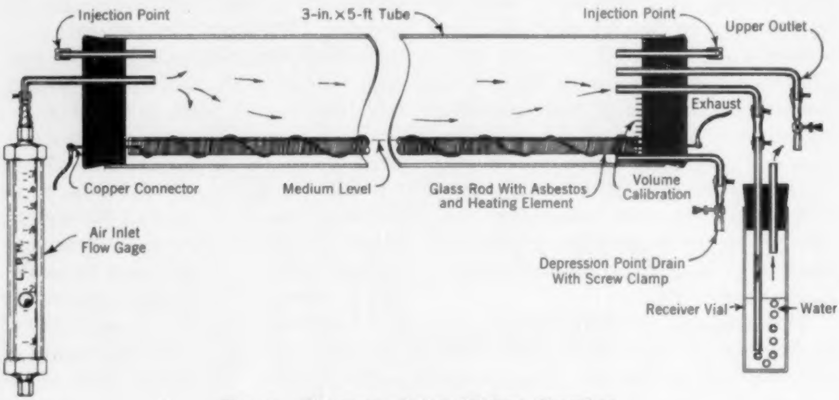

Fig. 2. Diagram of the Culture Chamber

The culture chamber was used to study the byproducts of actinomycete metabolism as the organisms developed. They were introduced into the chamber at the earliest (spore) stage of their life cycle.

* Model DU; a product of Beckman Instruments, Inc., Fullerton, Calif.

other available sources of organic carbon are far superior to sucrose. Also, organic nitrogen stimulates more colony development and enhances odor production. The techniques and equipment afforded an opportunity to study the aquatic forms in considerable detail and to evaluate them in regard to their requirements.

Chemical Byproducts

During the last 5 years experiments have been performed with the various types of equipment that can be used specifically to study the taste and odor compounds increased by the actinomycetes. Figure 2 represents a type of chamber that has been used during the past 2 years with unusual success. The major problem in using large culture chambers for metabolic studies has been to produce a sterile area for culture growth. This problem was solved by the type of construction shown in Fig. 2. In practice, a culture column was assembled from parts that had been meticulously cleaned. The chamber was mounted in a level horizontal position and electrical connections were made from a rheostat to the copper electrodes. The chromel element was heated until temperatures of 140–180°C were obtained on the interior of the column over several hours. Sterile air was continuously flowed through the column as it was heated so as to be certain that the rubber stopper inserts were sterilized. These were the areas that offered the greatest resistance to sterilization, and experience revealed that the continuous passage of heated air was the most successful technique to employ. Once the culture chamber was dry-sterilized, it was filled with a known volume of water and boiled 15–20 min each day for 3 days. The columns were of sufficient strength that pressures as high as 22 psi of steam could be obtained without danger to a laboratory worker. The slow method was preferred, however, because there were occasional invisible weaknesses in construction of the glass tubing. After the water had been sterilized as shown by negative cultures, it was heated to boiling, the current was suddenly cut off, and a flask of concentrated medium was simultaneously attached to the upper outlet of the culture chamber. All connections to the chamber were closed, including the airflow connection. As the partially evacuated chamber began to cool, the medium concentrate was aspirated into the interior. The total volume of liquid medium was so prepared as to amount to approximately 3,000 ml. The composition was variable but always contained potassium, sodium, calcium, sulfates, chlorides, nitrates, phosphates, bicarbonates, ammonium, organic nitrogen, and sources of organic carbon. On the first, second, and third days after the medium was added to the culture chamber, it was boiled for a period of 20 min. Samples of the medium were collected from the depression point drains and subcultured on the third and fourth days. If all cultures were negative, a spore suspension of 10 ml of an aquatic actinomycete was added at each injection point. In 24 hr the growth of the primary stages appeared as flocculant masses in the lower part of the chamber. Later the secondary stages were observed submerged in the medium, floating on the surface, or attached to the asbestos-covered core.

The purpose of the culture chamber was primarily the study of the byproducts of the metabolism of the actinomycetes as they developed. The com-

pounds that have been studied, which do not volatilize, but remain in the medium, do not appear to be of importance in producing the musty, woody, or earthy tastes and odors. Consequently, the equipment used served admirably for studies of these organisms and is adaptable to numerous problems involving plants or animals that produce volatile compounds.

Culture Chamber Investigations

The investigations on the aquatic actinomycetes so far have considered four of the eleven species described from the collection. Because documentation is incomplete on the taxonomy of the aquatic actinomycetes, a description will be given of the general characteristics of those studied. As mentioned earlier, the primary stages of the aquatic actinomycetes occur in submerged culture and are small and difficult to observe. Approximately 12 hr after the injection of a spore suspension, diffuse masses of primary stages could be discerned on the lower side of the culture chamber. As the masses became apparent, the exhausted air from the chamber became laden with volatile materials that had a fishy odor. After 24 hr the odor generally became grassy, and after 1–2 days it was haylike. At this time small flecks of the secondary stages developed around the periphery of the liquid medium or on the asbestos core. At this stage in development the types and intensities of the odors began to vary. Certain species produced swampy, marshy, or cucumber-like odors. Others gave off musty odors, and two of the species investigated did not give off marshy or musty odors, but rather produced earthy and woody odors. In 5–6 days after inoculation the secondary stages of the actinomycetes completely covered the liquid medium and core. As long as the mat remained moist in the chamber, the air flow delivered volatile organic compounds. The chamber operated 2–3 months, during which time substantial quantities of odor compounds were exhausted.

Numerous techniques have been employed in an attempt to concentrate the organic components elaborated by the actinomycetes. Cold water was used to concentrate the byproducts, and in that manner samples with a threshold odor of 2,000 were obtained. Extraction and drying of these samples indicated that the organic compounds occurred in concentrations of 0.14–0.8 ppm. The chemical nature of these compounds has been explored for several years. The components were divided in acid, basic, and neutral fractions and studied on the infrared spectrophotometer. Unfortunately, so many groups are present that accurate determinations are not possible. A few of the compounds have been identified microchemically. In general, however, these taste and odor compounds are so readily oxidized that derivative formation and purification are not successful techniques. Investigations with activated carbon have been performed with varying degrees of success. Carbon immersed in water and connected directly to culture chambers soon becomes saturated with odor compounds. If the carbon is kept cool, it appears to have greater accumulating capacity. When the odor compounds begin to flow through a carbon column that has been in service for several weeks, only small quantities of the compounds are adsorbed and it is therefore assumed that the carbon is saturated. In the desorption studies carbon samples were dried and extracted with chloroform, ether, or alco-

hol. The solvents were evaporated and gravimetric data were accumulated. The byproducts recovered by this technique were resinous and bore no resemblance to the original compounds insofar as odors were concerned. The extraction of wet carbon with the same solvents yielded greater quantities of organic compounds, but the dry residues, again, had none of the odor characteristics of the original material. It was concluded that these techniques were of little value in accurately determining the chemical nature of the compounds. There is one procedure that has been successful in the recovery of a portion of the organic compounds that had been adsorbed on carbon. This method is based on the use of heat and high vacuum for desorption, with subsequent collection in water at 0°C. Samples of carbon saturated with odor compounds were placed in large scrub bottles in an oven. The bottles were arranged so that extended flexible tubing protruded from the upper oven outlets. Polyethylene tubing connected the carbon containers to a scrub bottle in a freezer. A partial vacuum was placed on the bottle containing cold water and the carbon samples were slowly heated. As the temperature increased, the organic compounds were desorbed from the carbon and retained in the cold water. This is not a quantitative procedure, but it appears to be qualitative, if distinct odors are any criterion. Water samples prepared in this way may obtain threshold odors of 10,000 or more. Infrared analyses made on carbon disulfide, chloroform, and carbon tetrachloride extracts of water samples resembled analytical results obtained from chamber samples. There were variations in the tracings, which indicated that carbon did not release or adsorb certain groups of organic compounds, and that method of collecting odor compounds was ultimately abandoned. These results indicate that raw-water collections of odor compounds would yield inconclusive results and unrepresentative data.

A recent development (Fig. 3) has proved successful from the standpoint of concentrating the taste and odor compounds. The scrub bottle may or may not be used in the system, depending upon the study in progress. If it is desirable to remove the basic components, acid is added to the scrub bottle and the acid components are collected as frozen concentrates. In a similar fashion a basic fluid may be substituted in the scrub bottle, in which case the basic components may be collected. If it is desirable, the scrub bottle may be removed from the circuit and all the compounds frozen out. The sterile-air stream is dry when it enters the culture chamber, and in passing through an atmosphere above a liquid medium it not only acts as a vehicle for transporting volatile organic compounds, but it also becomes saturated with water. Thus, the frozen concentrate contains 60–98 per cent water, depending upon the amount of organic compounds being produced. If the exhausted air is directed through a scrub bottle containing inorganic compounds that are dehydrating but unreactive to the organic components, then almost pure crystals of the odor compounds may be obtained. All odor compounds do not freeze at −80°C, and it may therefore be necessary to use liquid oxygen to collect the total organic product. These organic compounds are subject to deterioration unless stored in airtight containers or preserved by refrigeration. The rapid oxidation of the odor components poses

many problems if analytical techniques require exposure of the compounds to room temperature and air.

Newer Analytical Methods

A number of investigators have used gas chromatography for the separation of organic compounds. If the kinds of compounds are known, the instrument serves a very useful purpose. Water mixtures are known to produce tailing in instrumental tracings, however, and this may lead to confusion in making positive identifications. As the odor compounds are unknown, it would seem that gas chromatography might not yield good results. Although the experiments in the use of this instrument have been only preliminary, it appears that the odor compounds may be separated and identified individually. This technique requires an adsorption tube of larger diameter and approximately 6-ft length. The adsorbent that has given the best results is brick dust. The chromatographic tube is usually connected to a culture chamber with a drying tube interconnecting both units. A helium source replaces the sterile air and the volatile organic compounds are swept from the culture chamber and through the drying unit to remove moisture and adsorbed onto the brick dust. This is not common practice, but, in this type of investigation, it appears to be a useful technique. Once the brick dust is saturated, the chromatograph tube is disconnected from the culture column and tied directly to the helium source. A gas collection chamber is set in place behind the

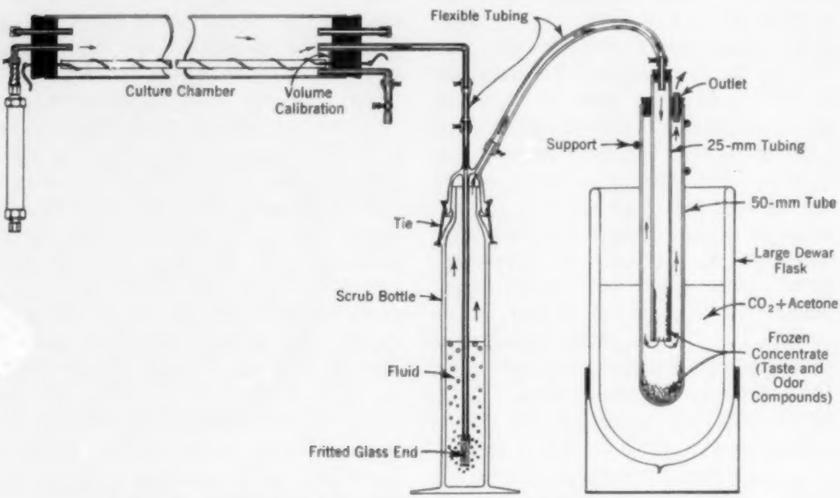

Fig. 3. Collection Apparatus for Taste and Odor Compounds

Volatile organic compounds are picked up by the air stream passing through the culture chamber. Because the air becomes saturated with water vapor it is dehydrated in the scrub bottle; it then passes into the Dewar flask, where the organic compounds are frozen and collected.

chromatograph and the unit is slowly heated. This procedure is another digression from the normal use of the equipment, but if the temperature is raised one degree at a time, fractions of the odor compounds are desorbed, recorded as tracings, and collected in the gas chamber for infrared studies. The method appears to have considerable promise; the organic products do not degenerate with the same rapidity as they do in the presence of air. As soon as the helium has operated for a few hours on any culture chamber, it is replaced by air. There is no differential in the production of the organic components produced by the growing actinomycetes. One difficulty in taste and odor research is that the identity of the taste and odor compounds must be judged solely on the basis of the presence or absence of odor. Organic solvents have not been used, because the volatility of an organic solvent and its acute odor would normally mask the odor of the organic compound and one would not know whether one were identifying the compound as it was when it produced the odor, or oxidized fractions, which might have no odor at all. Attempts have therefore been limited to analytical work on compounds whose odors are still available for identification. The method described above yields a variety of compounds that are separated at temperatures of 32–160°C. The characteristics of these chemicals will be described after more studies are completed.

Culture Column Studies

The culture chambers may also be used for studies of the organisms associated with the aquatic actinomycetes. Algae have been raised in the columns in order to study their odor-producing capacities. The genera investigated were *Asterionella, Cyclotella, Anabaena, Aphanizomenon, Eudorina,* and *Gleocystis.* It is not certain that these cultures were unispecific, but they were generally free of bacteria, fungi, and actinomycetes as shown by subcultures. The only apparent odors that could be identified from the algae alone were grassy, vegetable, and fruity. On two occasions the blue-green algae produced putrid odors when the medium became depleted in nutrients. In one culture chamber a growth of *Aphanizomenon* produced a strong earthy odor. Samples of the algae were removed, frozen, homogenized, and added to a general culture medium. In a few days actinomycetes developed and in mass culture produced the earthy odor.

No pure algae cultures that produced musty, woody, or earthy odors in discernible concentrations have been reared. When these odors are present, they appear to be caused by the presence of aquatic actinomycetes. The blue-greens furnish a remarkable substrate for the growth of actinomycetes, however, and are the best culture media available. Unfortunately, these algae are so complex in their own physiology that it would have confused the problem of nutrition to add additional unknowns. The same applies to the diatoms, particularly *Melosira* and *Cyclotella.* Doubtless other genera would serve as a culture medium, but those investigations should be reserved for algologists.

Another point of interest in culture chamber research has been the study of compounds that reduce or kill the actinomycetes. Results with copper show that a supply of available organic nitrogen renders copper ineffective (*8*). The reader will observe that

copper electrodes projected through the stoppers were tied to the heating units in the chambers (Figs. 2, 3). The wires were slowly corroded during column sterilization so that 2–5 ppm available copper was present in the medium at all times. The culture chambers have been used for the study of other compounds toxic to the actinomycetes and some very interesting observations have been recorded. For example, actinomycetes can tolerate formaldehyde in concentrations of 10 ppm and in 5–6 days completely metabolize it. They also metabolize phenols in greater concentrations, and this may slightly increase their earthy odor–producing capacities. As far as is known, these forms do not change their characteristic odor regardless of the source of their nutrition, although the intensity of the odor may vary. These observations bring up the subject of the relationship between the natural constituents of raw water and those that are considered pollutants.

The nutrition of the aquatic actinomycetes involves almost any pollutant that contains enough organic carbon and organic nitrogen to be utilized in their metabolic cycle.

It will undoubtedly be some time before anyone can draw definite conclusions concerning the organic compounds that naturally occur in water, since many pollutants undoubtedly are assimilated by actinomycetes and other organisms and utilized as a part of their regular metabolic cycles. It will be impossible to predict the organic compounds in water supplies until it is well established what chemicals are utilized by or produced by bacteria, algae, aquatic actinomycetes, and other organisms. Basic investigations in these fields should certainly be exploited before conclusions are drawn concerning the role of waste compounds in rivers, streams, and lakes.

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Impoundment and Water Quality

Panel Discussion

A panel discussion presented on Mar. 24, 1958, at the Southeastern Section Meeting, Atlanta, Ga.

Reregulation of Impounded Water, Chattahoochee River, Ga.—*Benjamin M. Hall Jr.*

A paper presented by Benjamin M. Hall Jr., Hydraulic Engr., Wiedeman & Singleton, Engrs., Atlanta, Ga.

THE USGS established a gaging station, designated as Chattahoochee River at Atlanta, Ga., on Paces Ferry Bridge, about 2½ mi upstream from the intake of the Atlanta Water Works, in August 1928. It operated it until December 1931, when it was temporarily discontinued. It was reestablished in November 1936 and has been operated continuously to date. Direct records at the station cover a period of about 23 years. Other stations on the Chattahoochee River in the vicinity of Atlanta have been operated continuously since 1903 and these records have been combined to represent the flow at Atlanta from 1903 to 1955, a period of 53 years. The drainage area at the Atlanta station is 1,450 sq mi.

Discharge and Flow Duration

For the period of direct record, extremes of discharge have been:

Maximum observed: 59,000 cfs on Jan. 9, 1946, gage height 28.0 ft

Maximum daily: 42,100 cfs on Jan 9, 1946

Minimum observed: 296 cfs on Sep. 30, 1954

Minimum daily: 340 cfs on Oct. 19, 1954, gage height 1.71 ft

The average discharge for a period of 21 years (through Sep. 30, 1955) is 2,534 cfs.

The USGS has prepared a duration curve of unregulated natural daily flows at Atlanta for the period 1909–47 (*1*). This curve has been extended by the author to include the 8-year period ending Sep. 30, 1955, and is summarized in Table 1.

Effect of Buford Dam

The Buford Dam, located on the Chattahoochee River about 48 mi upstream from the Atlanta Water Works intake, was begun by the US Corps of Engineers in 1950. This dam, for combined flood control, navigation, and power, will give a maximum power head of 150 ft, an average power head of 136 ft, and a power storage of more than 1,000,000 acre-ft with a drawdown of 35 ft. In addition, flood storage capacity above the level of maximum power pool will amount to more than 600,000 acre-ft. The drainage area at the dam site is 1,046 sq mi.

The maximum flood of record, 55,000 cfs, occurred in January 1946.

The power plant, which will be used for peaking operations, will consist of two units of 40,000 kw each and one unit of 6,000 kw, a total of 86,000 kw. It is understood that peak-load operation with maximum flows of about 8,000 cfs will generally be limited to about 6 hr during the day for 5 or 6 days per week. At other times the 6,000-kw unit will be operated normally at 500 cfs so as to provide a minimum flow of 600 cfs at Atlanta (the low-water inflow between Buford Dam and Atlanta is estimated at 100 cfs).

TABLE 1

Daily Flow Duration, 1909–55

Flows Equal to or Greater Than *cfs*	Duration *% of time*	Duration *days/avg. yr*
8,000	3.5	13
6,000	6.0	22
5,000	8.0	29
4,000	13.5	49
3,000	23.5	86
2,500	35.0	128
2,000	48.0	175
1,500	67.5	246
1,300	75.0	274
1,000	87.0	318
800	94.0	343
700	96.5	352
600*	98.0	358
500	99.0	361
400	99.65	364
300	99.93	365

* The average year flows have been equal to or greater than 600 cfs for 358 days; conversely, flows have been less than 600 cfs for only 7 days of the average year.

The dam was completed in February 1956 and storage commenced at that time. The power plant was recently completed and was scheduled to begin operating sometime during the summer of 1958.

Flood Control

The design flood has been estimated by the US Corps of Engineers as 279,000 cfs, or five times as large as the greatest flood of record. It is estimated that the flood storage afforded by Buford Reservoir is sufficient to absorb completely a flood of 55,000 cfs, such as occurred in January 1946, without outflow. The estimated maximum outflow for the design flood of 279,000 cfs would be 28,100 cfs.

The effect of this large flood storage will be to reduce greatly floods at Atlanta. These will not be entirely eliminated, as there are about 400 sq mi of uncontrolled drainage area between Buford Dam and the Atlanta Water Works intake. The USGS estimates that at Atlanta the "50-year flood" will be reduced by Buford Dam from 49,000 to 28,000 cfs, a reduction of 43 per cent.

Power Peaking During Average Flow

The US Corps of Engineers has prepared a chart, designated as chart No. 1, dated January 1951, showing the estimated typical flow pattern at Atlanta for power operation at Buford Dam during a full week of average flow conditions. Average flow from Buford Dam is calculated to be 1,920 cfs, plus an assumed base flow of 600 cfs from the intervening area between Buford Dam and Atlanta, a total of approximately 2,520 cfs. The minimum flow at Atlanta would be 600 cfs according to this chart. Chart No. 1 shows:

Average flow for 6 days per week with hourly flows varying from 700 to 6,700 cfs = 2,831 cfs

Average flow for 1 day per week with hourly flows varying from 600 to 700 cfs = 634 cfs

The average daily flow is summarized:

6 days @ 2,831 cfs	= 16,986 cfs
plus 1 day @ 634 cfs	= 634 cfs
Total for 7 days	= 17,620 cfs
Average	= 2,517 cfs

During average flow periods, it is apparent that on one day each week (Monday) the flow at Atlanta would average only 634 cfs.

Power Peaking During Low Flow

Another chart, designated as chart No. 2, has been prepared by the US Corps of Engineers to show the estimated typical flow pattern at Atlanta for power operations at Buford Dam during a full week of low flow conditions. Low flow from Buford Dam is calculated to be the prime flow of 1,600 cfs, plus an assumed base flow of 100 cfs from the intervening area between Buford Dam and Atlanta, a total of approximately 1,700 cfs. On this chart the minimum flow at Atlanta is also given as 600 cfs. Chart No. 2 shows:

Average flow for 5 days per week with hourly flows varying from 600 to 5,700 cfs = 2,136 cfs
Average flow for 2 days per week with hourly flows varying from 600 to 770 cfs = 652 cfs

The average daily flow is summarized:

5 days @ 2,136 cfs	= 10,680 cfs
plus 2 days @ 652 cfs	= 1,304 cfs
Total for 7 days	= 11,984 cfs
Average	= 1,712 cfs

These calculations show that during low-water periods there would be a 2-day "weekend trough" at Atlanta with an average flow of about 650 cfs. This weekend trough is estimated to begin about 10:00 PM on Saturday and end about 10:00 PM on Monday.

Chart No. 2 shows that weekday peaks would arrive at Atlanta (USGS gage at Paces Ferry Bridge) at 2:00 AM, and that minimum daily flows would occur about 10:00 PM each day.

TABLE 2

Discharge Tests

Time Period	Outflow cfs
Nov. 14, 1956	
prior to 8 AM	480
8–10 AM	4,100
10 AM–NOON	4,600
NOON–1 PM	4,000
1–5 PM	0
5–6 PM	2,500
6–7 PM	5,600
7–8 PM	4,600
8 PM–MIDNIGHT	500
Nov. 15, 1956	
MIDNIGHT–8 AM	500
8–10 AM	4,100
10–11 AM	4,600
11 AM–NOON	5,600
NOON–1 PM	3,000
1–5 PM	0
5–6 PM	2,500
6–7 PM	5,600
7–8 PM	4,600
8 PM–MIDNIGHT	450
Nov. 16, 1956	
after MIDNIGHT	450

Discharge Tests

At the request of the consulting firm of Weideman and Singleton, Engineers, Atlanta, the US Corps of Engineers

conducted tests of discharge from Buford Dam on Nov. 14 and 15, 1956, to simulate power flows during weekdays of a low-water period. The sluice gates at the dam were opened at 8:00 AM on Nov. 14 and, according to data furnished by the corps, were operated as shown by Table 2.

The average release from the dam from 8:00 AM on Nov. 14 to 8:00 AM on Nov. 15 was 1,670 cfs for the first 24-hr period; an average of 1,645 cfs was released during the second 24-hr period on Nov. 15–16. At the USGS gage in Atlanta, as a result of these releases, the maximum flow on Nov. 15 was 2,400 cfs, the mean flow was 1,760 cfs, and the minimum flow was 900 cfs; on Nov. 16 the maximum flow was 2,600 cfs, the mean flow was 1,800 cfs, and the minimum flow was 820 cfs.

Attention is directed to the fact that in these tests of simulated power flow there were two peaks each day, the first occurring just prior to noon and the second occurring from 6:00 to 7:00 PM. Between these peaks there was a 4-hr period (1:00–5:00 PM) with zero outflow. These tests cover weekday operations only and were not extended to show the effect of the weekend shutdown at Atlanta. The maximum release from Buford Dam during the November 1956 tests was only 5,600 cfs as compared to the maximum of 8,000 cfs under operating conditions.

TABLE 3

Stations at Which River Stage Was Traced

Station	River Mile Location	Mean Sea Level Elevation of Ordinary Low Water *ft*	Average Fall *ft/mi*
At mouth of Buford Tail Race	348.3	913.3	
Automatic recorder below Buford Dam*	348.1	913.0	0.2
USGS gage near Buford, Ga.*	345.8	908.7	1.9
US Weather Bureau gage near Norcross, Ga.*	330.8	879.8	1.9
DeKalb County water intake	325.4	865.2	2.7
USGS gage near Roswell, Ga.*	318.8	851.2	2.1
Morgan Falls Pool	312.6	847.4	
Morgan Falls Tail Race	312.6	797.6	8.6†
USGS gage, Atlanta, Ga. (Paces Ferry Road)*	303.0	752.5	4.7
Clayton sewage treatment plant, Atlanta, Ga.	300.4	743.0	3.7
Length of reach of river—*mi*	47.9		
Total fall in 47.9 mi—*ft*	170.3		
Average fall in 47.9 mi—*ft/mi*	3.6		

* Hydrographs of hourly discharge were prepared by the US Corps of Engineers at these stations and also at the Buford Dam sluice gates.
† From Morgan Falls Tail Race to USGS gage near Roswell, Ga.

In cooperation with the USGS, the Georgia Power Company, DeKalb County, and the city of Atlanta, the US Corps of Engineers has prepared hydrographs showing the river stage each hour during the test period at the points listed in Table 3.

With these data in hand, it is possible to trace the simulated power waves downstream from Buford Dam

to Atlanta and below. For example, the morning peak of November 14 traveled as shown in Table 4.

The travel time of the other three peaks was found to be similar. The total elapsed time from Buford Tail Race to the Clayton plant was:

Peak of 7:00 PM Nov. 14 arrived Clayton plant 12:30 PM Nov. 15—time elapsed, 17½ hr

Peak of 12:00 NOON Nov. 15 arrived Clayton plant 4:30 AM Nov. 16—time elapsed, 16½ hr

Peak of 7:00 PM Nov. 15 arrived Clayton plant 12:00 NOON Nov. 16—time elapsed, 17 hr.

weekdays follows the two-peaks-per-day pattern set by the November 1956 tests, daytime flows at the Clayton Plant would be materially increased and more sewage dilution would be provided.

In view of this, and the further fact that weekend conditions are shown only on chart No. 2, this chart has been adopted for reregulation studies.

Summary

On the assumption that Buford Dam will be operated as previously outlined the effect at Atlanta may be summarized:

TABLE 4

Distance Traveled by Morning Peak

Hour	Station	River Mile Location	Distance Traveled *mi*	Time Elapsed *hr*	Average *mph*
11:45 AM	Buford Tail Race	348.3			
6:00 PM	Norcross gage	330.8	17.5	6¼	2.8
10:00 PM	Roswell gage	318.8	12.0	4	3.0
MIDNIGHT	Morgan Falls Tail Race	312.6	6.2	2	3.1
3:00 AM	Atlanta gage	303.0	9.6	3	3.2
4:15 AM	Clayton plant	300.4	2.6	1¼	2.1
Totals			47.9	16½	2.9*

* Average.

These data show that travel time averaged 17 hr and that the morning peak leaving Buford would arrive at the Clayton plant about 4 AM, followed by the evening peak leaving Buford, which would arrive at Atlanta about noon.

The original 1951 US Corps of Engineers chart No. 2 (previously described) shows that the daily peaks would arrive at the USGS gage at Paces Ferry Bridge, Atlanta, at 2 AM and it is considered that the flow tests of Nov. 14 and 15 thoroughly confirm the basic weekday flow data in chart No. 2. If future Buford operation on

1. Minimum flows with Buford regulation would be 600 cfs at all times. The actual minimum daily flow at Atlanta for the period of record was 340 cfs on Oct. 19, 1954, but the estimated lowest average flow for a 5-day period in September 1925 was only 230 cfs. It has been shown in Table 1 that natural flows at Atlanta have been less than 600 cfs for an average of 7 days per year, or about 2 per cent of the time. The regulation afforded by Buford Dam during severe drought periods will be beneficial.

2. During weekend shutdowns flows would be less than 700 cfs on 1 day

each week (Monday) during average flow periods, and less than 700 cfs for 2 days each week (Sunday and Monday) during low periods. Table 1 shows that natural flows at Atlanta have been less than 700 cfs for an average of only 13 days per year, or about $3\frac{1}{2}$ per cent of the time. Table 1 also shows that the natural average flow of 2,500 cfs is available 35 per cent of the time, or that flows would be less than 2,500 cfs 65 per cent of the time, or 34 weeks per year. It is therefore considered reasonable to estimate that there would be at least 34 weekend shutdowns when flows at Atlanta would be less than 700 cfs for 1 or 2 days per week as a result of the Buford regulation. Such low flows would always prevail at Atlanta on Monday and sometimes on Sunday and Monday.

Buford regulation is thus shown to increase the number of days with flows of less than 700 cfs from 13 to 34 days per year, or about 250 per cent. Such regulation, resulting from weekend shutdowns at Buford, would decrease the amount of dilution afforded by the present natural flows of the river at the Clayton sewage treatment plant. The necessity for further reregulation above Atlanta is clearly indicated.

Reregulation

A minimum flow of 600 cfs is equivalent to 388 mgd. For average domestic, commercial, and industrial water supply use such a supply is considered adequate for a population of about 2,300,000. The population of the Atlanta metropolitan area will, it is estimated, be approximately 1,600,000 by 1980. Strictly from a treated water supply standpoint, reregulation will not be required in the near future but should be considered in the long-range plan.

The Atkinson steam plant of the Georgia Power Company, located about 1 mi below the intake of the Atlanta Water Works, uses approximately 430 mgd of raw water for cooling purposes, or 11 per cent more than the regulated minimum supply or 388 mgd (600 cfs). In future years as more water is diverted for domestic and commercial uses, the supply available for raw water use by future industries will be decreased. Unless minimum flows are increased by reregulation, the raw water situation may become critical. An adequate supply of raw water in the river, which would be available for process water, should attract future industries to the area.

Reregulation to provide more water for sewage dilution at Atlanta would be most beneficial. The main Atlanta sewage treatment plant (the Clayton plant), located on the Chattahoochee River, provides primary treatment of sewage but not complete treatment. The present natural flows are insufficient to provide enough dilution with the present sewage loads at low flow seasons. The operations of Buford Dam are estimated to create weekend troughs at the Atlanta sewage treatment plant which will provide flows of less than 700 cfs for 34 days in an average year, or 9 per cent of the time, and a minimum of 600 cfs. In addition, it has been shown that the Buford reregulation would create peak flows at the Clayton plant at about 4:00 AM when the discharge of sewage is relatively small. With adequate reregulation at some point above Atlanta the minimum flows during weekend shutdown periods could be increased and flows at other times could be released to suit the demands for sewage dilution, thus permitting a continuation of the present relatively inexpensive treatment methods.

Storage for Reregulation

To provide complete reregulation, so that a uniform low water flow of 1,700 cfs could be maintained at Atlanta, it is estimated that a storage of 5,000 acre-ft would be required. A continual flow of 1,700 cfs at Atlanta would be equivalent to a supply of 1,100 mgd which is considered adequate for a population of more than 6,000,000. Such complete reregulation is not considered necessary in the near future, but it would be feasible to obtain such a supply of water when needed by constructing a reregulating reservoir with a capacity of 5,000 acre-ft or more.

It is the author's opinion that adequate reregulation for sewage dilution after partial treatment for a period of about 15 years can be provided by a storage of 3,200 acre-ft. One acre-foot amounts to 43,560 cu ft or 326,700 gal of water, and 3,200 acre-ft would be 1,045 mil gal. A reservoir with a capacity of 3,200 acre-ft may be visualized as having an average area of 400 acres with a depth of 8 ft. Such a reservoir could be filled or emptied in 24 hr by a flow of 1,600 cfs, or filled or emptied in 12 hr by a flow of 3,200 cfs. It is thus seen that 1 cfs flowing 12 hr is equivalent to 1 acre-ft.

It has been shown that it takes a power wave about 4 hr to travel from Morgan Falls to the Clayton plant. Assuming that a storage reservoir with a capacity of 3,200 acre-ft was constructed at or near the Morgan Falls Dam, water released from Morgan Falls at 6:00 AM would reach the Clayton plant about 10:00 AM. In the analysis of flows into such a reservoir and the reregulated flows released from the reservoir, shown in Table 5, the flow data shown on the US Corps of Engineers chart No. 2 for a full week at low water stage has been used, on the assumption that the flows shown on chart No. 2 would arrive at Morgan Falls 4 hr earlier. For this typical week of low water flow conditions both the inflow and the reregulated outflow have been averaged for 12-hr periods, day flows being considered as the average flow from 6:00 AM to 6:00 PM, and night flows being the average flow from 6:00 PM to 6:00 AM.

A summary of Table 5 shows:

Average flow each 12-hr night period = 1,050 cfs
Average day flow, 12 hr Monday = 2,470 cfs
Average day flow, 12 hr Tuesday = 3,100 cfs
Average day flow, 12 hr Wednesday = 3,150 cfs
Average day flow, 12 hr Thursday = 3,150 cfs
Average day flow, 12 hr Friday = 2,650 cfs
Average day flow, 12 hr Saturday = 1,050 cfs
Average day flow, 12 hr Sunday = 1,050 cfs.

Attention is directed to the fact that in Table 5 all flows are *average* flows. The actual outflows may be varied as desired. For instance, with an average daytime regulated outflow of 3,150 cfs, it would be possible to operate:

4 hr @ 2,150 cfs
4 hr @ 4,150 cfs
4 hr @ 3,150 cfs
Average 12-hr flow, 3,150 cfs.

Reregulated outflows at night could also be varied in any number of desired combinations, but it is suggested that a minimum flow of at least 700 cfs be maintained at all times. This would provide about 450 mgd at the Atlanta Water Works intake.

The reregulated outflow in Table 5 during weekends is shown to average 1,050 cfs from 6 AM Saturday to 6 AM Monday (48 hr). However, if 700 cfs is determined to be sufficient for reregulated night outflows, flows for this 2-day period could be varied as desired, and might be:

Saturday (day), average 1,700 cfs for 12 hr
Saturday (night), average 700 cfs for 12 hr
Sunday (day), average 1,100 cfs for 12 hr
Sunday (night), average 700 cfs for 12 hr
Average, 1,050 cfs for 48 hr.

Storage is also available at different sites above Atlanta:

1. *Vinings dam.* This proposed dam will be 43 ft high at the foot of Long Island, Ga., and will back water to the tail race of Morgan Falls Dam. This dam site is owned by the Georgia Power Company and surveys and estimates of cost have been made by the US Corps of Engineers and others. The estimated areas and storage capabilities are:

Full pond area = 800 acres
Estimated area with 10-ft drawdown = 480 acres
Average area = 640 acres
Storage capacity with 10-ft drawdown = 6,400 acre-ft.

Such storage would be adequate for complete reregulation.

The US Corps of Engineers estimated the cost of this dam, as of 1937–39, to be $2,105,000, exclusive of powerhouse, power plant, lands and relocations, and interest during con-

TABLE 5

Analysis of Operations of Proposed 3,200 acre-ft Reregulating Storage Reservoir

Period	Average Flow—acre-ft*		Storage—acre-ft*	
	Inflow	Reregulated Outflow	Amount Added or Released	Balance at End of Period
Monday night, begin 6 PM				0†
6 PM Monday–6 AM Tuesday (night)	3,280	1,050	+2,230	2,230
6 AM–6 PM Tuesday (day)	870	3,100	−2,230	0
6 PM Tuesday–6 AM Wednesday (night)	3,540	1,050	+2,490	2,490
6 AM–6 PM Wednesday (day)	1,110	3,150	−2,040	450
6 PM Wednesday–6 AM Thursday (night)	2,970	1,050	+1,920	2,370
6 AM–6 PM Thursday (day)	1,060	3,150	−2,090	280
6 PM Thursday–6 AM Friday (night)	3,720	1,050	+2,670	2,950
6 AM–6 PM Friday (day)	1,180	2,650	−1,470	1,480
6 PM Friday–6 AM Saturday (night)	2,770	1,050	+1,720	3,200‡
6 AM–6 PM Saturday (day)	860	1,050	− 190	3,010
6 PM Saturday–6 AM Sunday (night)	650	1,050	− 400	2,610
6 AM–6 PM Sunday (day)	680	1,050	− 370	2,240
6 PM Sunday–6 AM Monday (night)	670	1,050	− 380	1,860
6 AM–6 PM Monday (day)	610	2,470	−1,860	0†
7-day Average	1,712	1,712		

* An acre-foot is equivalent to 1 cfs flowing for 12 hr.
† Empty.
‡ Full; the reservoir would be completely filled by 6 AM Saturday.

struction. The average *Engineering News-Record* index for the 1937–39 period was 235, as compared to the present index of 705. On this basis, the approximate present estimated cost would be $6,315,000 for the dam only.

2. *Morgan Falls dam.* Several conferences have been held with officials of the Georgia Power Company, the owner of the present dam, as to the possibility of creating additional storage at Morgan Falls Dam by the construction of 8-ft taintor gates on top of the dam for reregulating purposes. According to the engineers of the Georgia Power Company, these 8-ft taintor gates would provide a storage of 3,200 acre-ft above the crest of the concrete dam, and the estimated cost of the development would be between $850,000 and $1,000,000. The beneficial reregulating effect of a 3,200-acre-ft reservoir has been explained previously.

3. *Roswell site, 15-ft dam.* No detail surveys have been made of Roswell site in the field. From information made available by the US Corps of Engineers it appears that a low dam consisting essentially of a concrete sill across the river and 15-ft taintor gates would develop about 5,500 acre-ft of storage with 10-ft drawdown, which would be sufficient for complete reregulation of the river at 1,700 cfs but would not provide excess capacity. There might also be some silting in the upper reaches of the reservoir. An approximate estimate of cost of such a reregulating dam, based on available data, is about $2,500,000, exclusive of land and flood rights which the author understands are owned by the Georgia Power Company. It appears that about 1,200 acres would be involved in flood rights.

4. *Roswell site, 24-foot dam.* A dam 24 ft high at the Roswell site consisting of a concrete base and 15-ft taintor gates should provide about 11,000 acre-ft of storage with 10-ft drawdown, or 7,500 acre-ft storage with 6-ft drawdown. This capacity would be adequate for any reregulation desired and could be operated to prevent silting in the effective top 6 ft. The cost of the higher structure would not involve excessive cost since the cofferdam and taintor gate cost would be the same. The principal additional cost would be in the mass concrete in the base. An approximation of the cost of this dam, based on available data, is $3,000,000, exclusive of land and flood rights which the author understands are owned by the Geor-

TABLE 6

Costs and Capacities of Proposed Storage Projects

Project	Storage Capacity *acre-ft*	Cost—$	
		Total*	Per Acre-Foot of Capacity
Vinings Dam	6,400	6,315,000	987
8-ft taintor gates on Morgan Falls Dam	3,200	1,000,000†	312
15-ft Roswell Dam with 10-ft drawdown	5,500	2,500,000	455
24-ft Roswell Dam with 6-ft drawdown	7,500	3,000,000	400

* Exclusive of land and flood rights, except as noted.
† Approximate total cost including land and flood rights.

gia Power Company. Approximately 2,000 acres would be involved in flood rights.

Table 6 summarizes the capacities and costs of these four projects.

Power installations have been purposely omitted in the plans for the Vinings Dam and the two dams of differing height proposed at the Roswell site and no power studies have been made. If power installations are included the cost to be allocated to reregulating storage might be materially changed.

If a reregulating reservoir is constructed at any point between Buford Dam and Atlanta it is considered essential that a firm agreement as to when water would be released and how much would be released be made by the city with the party operating the reregulating reservoir, as well as with various riparian owners downstream from the development. It would appear that there should be no conflict of interest between the city and the party operating the reregulating reservoir, if it is decided to use the released water for generating power. Peak daytime power is more valuable than run-of-stream or nighttime power and it appears entirely feasible to coordinate the power release with the demand for dilution at the Clayton plant, where sewage flows are heavy from 10 AM to 10 PM on weekdays.

Present Project Status

On Sep. 6, 1957 the city of Atlanta and the Georgia Power Company entered into a formal agreement for the construction and operation of the 8-ft taintor gates on Morgan Falls Dam. This agreement provides:

1. The city will contribute 50 per cent of the cost, the city's contribution not to exceed $500,000.
2. The Georgia Power Company will construct the improvements and bear 50 of the cost.
3. Title to Morgan Falls Dam, upon completion of the work, shall be held by the Georgia Power Company.

The city and the company have also made a satisfactory agreement as to scheduling releases of storage water from Morgan Falls Reservoir for the mutual benefit of the city of Atlanta and the Georgia Power Company. Application to the Federal Power Commission for a license for redevelopment of the Morgan Falls project has been filed by the Georgia Power Company, but the license has not yet been granted.

Reference

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Effect of Impoundment on Downstream Water Quality, Catawba River, S.C.—*Robert S. Ingols*

A paper presented by Robert S. Ingols, Head, Dept. of Applied Biology, Georgia Inst. of Technology, Atlanta, Ga.

The effect of the lakes above the large power dams of the Tennessee Valley Authority upon the quality of water downstream (*1*) and also some of the effects that this water quality has upon downstream use are interesting to consider.

The assurance of a steady, adequate quantity of water below a storage reservoir or power dam is very desirable

for a city relying upon a downstream river for its water supply. The fact that there is a large variation in volume twice daily as a result of the power wave does not affect the operation of the river pumping station appreciably. Dry weather also does not affect the minimum flow values from a power pool greatly because the use of water for the power surge is discontinued in order that the minimum flow requirements may be maintained.

where only the yearly increment of leaves, dead algae, and other natural debris causes a loss in DO of the hypolimnion (lower stratum of water).

The normal changes in water quality have been discussed by Churchill (*3*). Observations of downstream water quality made during four successive summers by the author have indicated, however, that periodic explosive deterioration in water quality occurs in old reservoirs. Wide variations in water quality have occurred in the Catawba River at Rock Hill, S.C., below a 60-year old reservoir dam. These variations are summarized briefly in Table 7.

TABLE 7

Observations on the Catawba River

Period	Manganese Volume	Fish Kills	Comments
1954	None reported	None reported	Very dry season; heavy growth on exposed flats*; no data on DO concentration
1955			Normal rainfall; lowest DO concentration in May
May		Heavy	
June	Heavy		
July	Light		
August	Light		
1956			Normal rainfall; lowest DO concentration 1.1 ppm on Aug. 5
August	Light	Light during high flow	
1957	None reported	None reported	Wet summer; lowest DO concentration 2.8 ppm on Sep. 9

* Observed on the Tennessee Valley Authority lake, Douglas Reservoir, below Dandridge, Tenn.

Seasonal Changes in Water

Water quality and power dam operation is nevertheless of interest to the water plant operator and the city engineer with sewage and industrial waste problems. Most of the data published (*2*) have indicated that artificial lakes and reservoirs a few years after their creation reach a condition of stability that insures good water downstream. After the initial effect of the submerged organic matter has disappeared the water downstream from a nonpolluted inlet supply improves to the point

The data of this table indicate that in 1954, a very dry season, the downstream water quality was reasonably good. However, when a visit was made to Rock Hill in the early summer of 1955, several unexpected observations were made. At the water plant when the light switch to the sight well was flicked on and off several times the visitors were unable to see the bot-

tom any better with the light on than with it off. It was discovered that the walls were so black that the light could not be observed, even though it was working properly. It was realized that the walls were coated with manganese dioxide. The problem of manganese in the filtered water was solved when copper ion catalysis for prefilter oxidation was instituted, as reported elsewhere by Chambers and the author *(4)*. This treatment was the result of observations on the water plant at Newnan, Ga., and was reported several years ago by Futral and the author *(5)*.

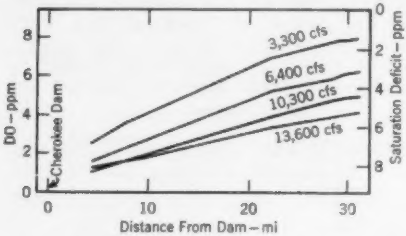

Fig. 1. Effect of Reaeration

The effect of the reaeration of the Holston River, Tenn., is shown. Cfs values indicate rate of flow. The reaeration took place below the Cherokee Dam during July and August 1957. (Taken from Ref. 3.)

A trip was made up and down the Catawba River to find possible paths of approach to the river for sampling and to learn how much damage the large industrial waste deposits from Rock Hill were causing in the river. It was learned that two or three fish kills had occurred in the spring of 1955, during May and June; fish kills at that time of year were very difficult to understand.

In August 1954 the author had been asked to examine a waste situation in Dandridge, Tenn. A small mountain stream flowed from this town into the headwaters of Douglas Reservoir, or more accurately, it flowed from the town through a channel in the dry bottom of the lake area. The pool level at the time of observation was at an elevation of 960 ft and the maximum possible elevation was 1,000 ft; the maximum elevation that spring had been 984 ft, leaving a large area not submerged for the second consecutive summer. The exposed mud flats were covered with a lush, green vegetation about 2 ft high and extending over a great many acres. A similar growth could have developed on an arm or area of the Catawba River lake bed. The lake was low in 1954, for there were only small power waves during the period of a state water pollution control authority study of the river in 1954, as seen from flow values and hydrographs.

The hypothesis is that the lush vegetation on the exposed flats of a power dam lake bottom during a dry year is responsible for an excessive organic-matter load in the hypolimnion when the vegetation is submerged by water after the dry season. The presence of the organic matter will remove all of the DO of the hypolimnion as soon as it forms. This would explain the high manganese content of the Catawba River in the spring of 1955 and also the low DO in the river, as indicated by the fish kills during the same period. The absence of fish kills and manganese in the springs of 1956 and 1957 would also fit this hypothesis because the mud flats were not exposed to permit a similar growth in 1955 or 1956 when there was normal rainfall. The fact that the lowest DO (and the appearance of manganese in the Rock Hill water) occurred in early August 1956, 3 months later than the low point

for 1955, indicates that much less organic matter was present in the lake in 1956 than in 1955. The fact that the date of lowest DO was later in 1957 than in 1956 and the absence of manganese in 1957 indicates that there was even less organic matter in 1957 than in 1956.

The first part of this article has pointed out two major problems of importance to cities or to industries which use river water below a power

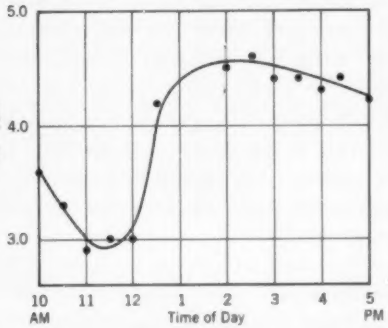

Fig. 2. DO Concentration During Power Wave

The DO concentration during a power wave is shown. These observations were made from a single point, 3 mi below the source of pollution, during the early stages of the power wave. The sharp dip in the curve shows the effect of the slime or sludge scour.

impoundment. At times there may be manganese present in the water. During the same period there may be a low DO content which will greatly reduce the capacity of the river to absorb pollution without serious fish kills and trouble with state regulatory agencies. It has also been pointed out that these problems are most severe during a year which follows an extended drought, though not necessarily during the dry season itself.

Short-Term Changes in Water

Superimposed on the long-term variations in water quality below a power dam are the daily changes in quality caused by the changing quantity of water due to peaking operations. When samples for DO are taken from one spot in a river below a power dam, first at low flow and then at high flow, the DO has been observed to fall from 3.8 to 2.7 ppm, as previously reported by the author (*6*). Much more extensive data have been obtained by Churchill (*3*) and a portion of his data is shown in Fig. 1.

These data directly refute the usual concepts of the BOD–oxygen-sag relationship by showing that increasing flow brings increasing severity of damage. In the usual concept, increased flow should bring increased dilution and a higher DO from the dilution. This is the reason that it has been claimed that the equivalent BOD load of a power dam increases with increasing flow. By extrapolating the curves of Fig. 1 to the dam site, it can be seen that there would be a deficit of almost 9.0 ppm DO at the dam. To create this deficit by a sewage BOD approximately 860,000 people would be required at a flow of 3,300 cfs; at a flow of 13,600 cfs 3,500,000 people would be necessary. This indirect population equivalent indicates the magnitude of the pollution problem which the power dam creates. There are no cities with this large a population in the Southeast and very few other industries in the area with a similar pollution load.

The BOD of the river water above the source of pollution does not change with or during the power surge, but the lower reaeration of the greater depth of water (there is a 20–30-in. rise on the Catawba River during the power surge) decreases the DO value

at a given point downstream. It is possible that the product of the total volume and the DO concentration will be larger at high flow than at low flow. However, regulatory agencies and fish do not understand this relationship, for only absolute DO values are of interest in codes of river quality, because of the desire to protect fish which are sensitive to the DO.

Besides the lower DO value occurring in the power wave above a source of pollution, there are two other damaging factors which occur below the point of pollution from the high flow of the power wave.

1. Sludges or slimes, or both, collect on the stones or in quiescent areas of rivers during low flow and are lifted from the bottom of the river during the first surge of the power wave and suspended in the flow. This suspension of the organic matter or slime causes a further drop in the DO value. During this period of low DO value fish are seen to die. Generally, the scouring of the sludges or slimes, or both, is completed in less than an hour. Figure 2 shows a curve of DO against time, at a single point of observation during the early portion of a power wave. In this case, too, it is unfortunate that the fish do not know that the slime scour is only a temporary situation. When the DO is low enough above the point of pollution addition then the 2.0–2.5-ppm sag in DO is enough to cause the fish real trouble. Thus, on the day of an observed 2.7 ppm DO above the point of pollution the sag in oxygen was enough to bring the DO value to 0.5 ppm in one portion of the river and fish were seen to struggle for air.

2. The high flow reduces the lateral mixing of the pollution load from a point source to such an extent that it has been observed to increase the damaging effect of the pollution, because the pollution is concentrated in such a small portion of the river. Thus, fish have been seen to die only in the polluted portion of the river when there is an overall DO content of less than 1.0 ppm.

When the point of pollution introduction is far enough downstream from the dam so that reaeration of the water is completed before the water reaches the point of pollution introduction, then the major factor of interest becomes volume of flow in the river in relation to the quantity of pollution to be added. This problem is discussed in greater detail in the article in this panel by Hall.

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Effect of Impoundment on Downstream Water Quality, Roanoke River, N.C.—*Frederic F. Fish*

A paper presented by Frederic F. Fish, Biologist, USPHS, Dept. of Health, Education, and Welfare, North Carolina State College, Raleigh, N.C.

The Roanoke River is an interstate stream rising along the eastern slope of the Appalachian Mountains in Virginia, flowing generally to the southeast, and discharging into Albemarle Sound in North Carolina. The river reach under consideration in this article is restricted to the lower 179 mi of the river between Kerr Dam and the mouth. Over this section the Roanoke River traverses the gently rolling topography of the Piedmont Plateau for 30 mi. Between the river Miles 150 and 129 it crosses the eastern escarpment of the Piedmont into the broad, flat Coastal Plains. Within this area the watershed is very narrow and no tributaries of significance join the main stem below Kerr Dam. Through the Piedmont the river gradient averages about 1.5 ft/mi. As the river crosses the fall line the gradient steepens to approximately 6 ft/mi between Miles 150 and 129. Below Mile 129, the gradient averages slightly less than 0.2 ft/mi across the wide Coastal Plains. The Roanoke River carries more water, by far, than any other river in North Carolina, the annual flow through the state averaging about 8,500 cfs. By the same token, it also possesses the greatest latent possibilities for development of any river in the state.

Impoundments

During the past decade two impoundments have been constructed on the Roanoke River within the area described. The first, the John H. Kerr project, was constructed by the US Corps of Engineers at Mile 179. Closed in 1952, the Kerr Dam impounded a lake of 2,800,000 acre-ft capacity, 39 mi long, with a normal depth under power operation of 110 ft. The Kerr project powerhouse has a generating capacity of 204,000 kw and is used for the production of peak power.

In 1955, the Virginia Electric and Power Company completed its Roanoke Rapids project at River Mile 137: a 100,000-kw power plant supplied by an 85,000-acre-ft impoundment, 9 mi long, with a normal depth of 65 ft. Like the Kerr project, the Roanoke Rapids project is used for the generation of peak power and is responsible for reregulating the river to maintain certain minimum flows downstream, as stipulated in its FPC license.

Waste Discharge

Immediately below the Roanoke Rapids project is an area of intensive water use. At the downstream end of an 8,000-ft excavated tailrace of the project, about 560 mil gal of river water are pumped each month to supply process water for a large pulp and paper manufacturing plant. Immediately below the pulp plant water intake a series of four major outfalls within a 1-mi reach discharge raw municipal and industrial wastes directly into the river. The particular wastes discharged are principally from pulp, paper, and textile plant opera-

tions, and domestic sewage. These wastes have a common characteristic: a high oxygen demand for stabilization. The wastes from the four outfalls, according to records published by the Division of Water Pollution Control of the state board of health in 1953, had a combined population equivalent of 574,000. A waste treatment program currently is being effected but, even after satisfactory facilities have been provided, carrying and absorbing organic wastes must remain an important function of the river. The 8-mi reach of the Roanoke River between the Roanoke Rapids project tailrace and Mile 129 traverses the steepest gradient of the fall line and provides the only rapids in the river now accessible to migratory fish. This reach constitutes the major spawning grounds for the entire Albemarle Sound striped bass population, a species indigenous to North Carolina waters. The striped bass runs to the Roanoke River spawning grounds were estimated to contain 164,200 fish in 1956 and 169,600 in 1957. Most of the striped bass have completed spawning and moved out of the river before the onset of the critical summer water conditions.

Effect of Impoundments

In view of the current and great potential use of the Roanoke River protection of water quality is a matter of paramount concern to the state of North Carolina. Recently, the Division of Water Pollution Control classified best use of the Roanoke River from the Roanoke Rapids projects tailrace to Albemarle Sound as fishing which requires, among other stipulations, the maintenance of at least 4 ppm DO at all times.

TABLE 8

Effect of Impoundment of Roanoke River Upon Downstream DO Concentrations

Sampling Results	Status of Projects		
	Post-Kerr, Pre-Roanoke Rapids (1953)	Post-Roanoke Rapids, 1st yr (1955*)	Post-Roanoke Rapids, 2nd yr (1956)
Number of samples	47	30	43
DO concentrations:			
Mean—*ppm*	7.90	5.35†	5.36
Standard deviation—*ppm*	0.49	1.01	0.79
Coefficient of variation—%	6.15	18.90	14.73
95% Confidence limits—*ppm*	6.90–8.90	3.30–7.40	3.80–6.90
Observed range—*ppm*	7.00–9.00	2.90–6.90	3.20–6.40

* Data by cooperation of Halifax Paper Co.
† Excludes four samples collected under atypical river conditions accompanying hurricane runoff.

The DO content of the Roanoke River water at the North Carolina Highway 48 bridge—which crosses at Mile 135—has assumed progressively greater significance during the past few years. This bridge comprises a check point for quality determinations on the river water as it leaves the Roanoke Rapids project and is withdrawn for industrial use. The bridge is immediately above the four major outfalls and hence also provides a sampling station for water quality data

before wastes are discharged into the river. The DO determinations made at the Highway 48 bridge during the past few years reveal that impoundment of the river has produced a different type of DO pattern than develops in a natural stream and one which complicates utilization of the full potential of the Roanoke River to absorb organic wastes.

The changes that have occurred in the Roanoke River DO pattern following impoundment are best indicated by the summarized data in Table 8, which were collected at the Highway 48 bridge. Extensive DO data also were collected during the summer of 1957 but these are not yet available.

The data on which Table 8 is based indicate that impoundment within the Kerr Reservoir made relatively insignificant differences in the DO concentrations obtained at the Highway 48 bridge, 44 mi downstream. The Kerr project releases water of very low oxygen content during the summer months when stratification develops within the reservoir. Reaeration provided by the 44 mi of natural stream bed between Kerr Dam and the Highway 48 bridge that existed prior to the construction of the Roanoke Rapids project was adequate to restore most of the oxygen deficiency of the Kerr project discharge. In general, the 1953 DO determinations at the Highway 48 bridge, which ranged between 84 and 104 per cent of saturation, reasonably approximated the variations that may be expected in a natural, unpolluted river.

It is apparent from a comparison of the 1953 data with those of 1955 and 1956 that impoundment within the Roanoke Rapids Reservoir has sharply reduced the average summer DO concentration of the river water. Under the pre-Roanoke Rapids project river conditions of 1953 the mean summer DO concentration was 7.90 ppm and the spread of the data about the average was indicated by a coefficient of variation of 6.15 per cent. During the first year of impoundment within the Roanoke Rapids Reservoir the mean summer DO concentration at the Highway 48 bridge was 5.35 ppm and the coefficient of variation was 18.90 per cent. The second year of impoundment produced an average DO concentration of 5.36 ppm and the coefficient of variation was 14.73 per cent. Thus, it appears that impoundment within the Roanoke Rapids Reservoir produced a decrease of about 2.50 ppm in the average summer DO concentrations in the critical water-use area immediately below the project. Occasional samples indicated that the drop in DO concentrations recorded at the Highway 48 bridge stemmed in large part from the failure of the reservoir to provide the same degree of reaeration formerly acquired in the inundated natural stream channel. The residual effects from deoxygenation in Kerr Reservoir, formerly eliminated in the natural stream channel, now extend through the Roanoke Rapids Reservoir and reach the Highway 48 bridge.

The implications of the loss of downstream DO following impoundment are quite apparent in an area where organic waste carriage and absorption are an essential function of the river. However, the importance of the much wider variations in summer DO concentrations should not be overlooked as an undesirable consequence of river impoundment. Utilization of the full potential of the river to absorb organic waste is greatly complicated, for available oxygen concentrations are no longer a direct function of river dis-

charge. The quantity of organic wastes that can be safely discharged into the river will be quite different if, at the point of waste loading, the river water contains 3.80 instead of 6.90 ppm of oxygen per acre-foot of water (these are the 95 per cent confidence limits for DO per acre-foot in the 1956 data). In a river the size of the Roanoke, with an average summer discharge approximating 4,000 cfs, variations in DO concentrations of this magnitude involve differences in daily oxygen concentrations amounting to more than 25,000 ppm. The variation in oxygen content of water may be just as great within days as between days.

Pollution Abatement

The regulated release of pulp mill wastes has been recommended as an appropriate pollution abatement measure. Under prevailing circumstances the bases for releasing these wastes must include the DO content of the river water, as well as river discharges, if the maximum value of the river for absorbing these oxygen-consuming wastes is to be realized. Waste loading can be safely increased when the oxygen content is high but it must be curtailed when the oxygen content is low, even though the river discharge is identical under both circumstances.

At present it appears that the fluctuating DO concentrations created by impoundment of the Roanoke River have introduced somewhat of a dilemma between releasing the wastes on the basis of river discharge integrated with results obtained from constant monitoring for DO, or on the basis of river discharge with an assumed oxygen content fixed at some arbitrary value, presumably the mean. Oxygen monitoring coupled with river discharge offers a safe, but expensive and complicated method. The release of wastes on the basis of river discharge with an arbitrarily fixed oxygen content would be much simpler. It would, however, result in unutilized oxygen assets of the river when the actual oxygen concentrations present exceeded the arbitrary value. Also, on this basis stream classification standards downstream might be jeopardized when the oxygen content of the river water fell below the assumed value.

Summary

Impoundment of the Roanoke River has exerted dual effects upon DO concentrations immediately downstream: average summer DO concentrations have been significantly reduced and variations in DO concentrations are much wider. Each effect has its serious implications in a local situation in which the river must absorb a heavy organic waste loading before it has an opportunity to recover from the degrading effects of impoundment.

Estimating Reservoir Yields on a Digital Computer

Samuel Jacobson, Everett L. MacLeman, and Richard E. Speece

A contribution to the Journal by Samuel Jacobson, Director of Lab., New Haven Water Co.; Everett L. MacLeman, Assoc. Prof.; and Richard E. Speece, Asst.; both of the Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

ESTIMATIONS of the safe yields of impounding reservoirs may be obtained through various procedures, the choice of which depends upon available data. Some of the methods commonly used are the Rippl or mass-diagram method (*1*), Allen Hazen (*2*), Desmond Fitzgerald (*3*), New England Water Works (*4, 5*), and direct statistical analysis. A comparative study of these methods has been published by one of the authors (*6*).

A typical problem in yield for which these methods do not provide the desired information is found in the Mill River system of the interconnected supplies of the New Haven Water Co. This system has a watershed of 37.7 sq mi and a usable storage in Lake Whitney of only 258 mil gal. Based on this storage, the standard procedures mentioned above indicate a safe yield of approximately 7 mgd for a 95-per cent dry year. The slow sand filter now in operation has a plant capacity ranging from 10 to 12 mgd. During the 40-year period from 1918 to 1957 the average flow at the Whitney Dam was 42 mgd. The minimum annual average flow for the period was 26 mgd in 1930, and a maximum of 77.2 mgd occurred in 1953. Although this water must be filtered (*7*), its potential for future use is considerable.

One possible procedure for increasing the use of this supply would be to increase the use of the other reservoirs of the company during dry periods. Drawdown of a reservoir beyond the capacity of its watershed to replenish it during the runoff period is usually avoided. Lake Whitney, owing to a large watershed and small storage capacity, is naturally the first to fill and overflow; more extensive use of filtered water from Lake Whitney would permit a reduction in the amount of withdrawal from the other reservoirs. The capacity necessary for any proposed filtration plant at Lake Whitney for such an operational procedure would require study from the standpoints of engineering and economics.

An overly large filtration plant with most of its capacity idle during dry periods would be uneconomical. This would also be true during years with high runoff, when the other reservoirs would generally be full. The first step in the investigation of such a problem is to study the effects of filtra-

tion plants of various capacities on yield. By using past records, one may compute available runoff or the amount of water that plants of various sizes would have produced had they been operating at full capacity during the entire period. This full yield would provide an indication as to the extent to which a plant would be in use. In this study the average full yield for the recorded period and for individual years was obtained for plants with 20–60-mgd capacities.

Computer Analysis

The data used cover 40 years of the Mill River records maintained by the New Haven Water Co. Several weeks are required to determine the average full yield for a filtration plant of a specific capacity. As indicated previously, a complete study would

TABLE 1

Full Yield From a Plant With a Capacity of 30 mgd

Yield *mgd*	No. of Days Available			Full Yield *mil gal*	
	1918–57	Avg Year	Driest Year (1930)	Avg Year	Driest Year (1930)
0	138	3.4	2	0.0	0
1	25	0.6	1	0.6	1
2	37	0.9	4	1.8	8
3	50	1.3	5	3.9	15
4	111	2.8	5	11.2	20
5	163	4.1	14	20.5	70
6	253	6.3	25	37.8	150
7	323	8.1	31	56.7	217
8	358	8.9	21	71.2	168
9	290	7.2	18	64.8	162
10	312	7.8	16	78.0	160
11	253	6.3	8	69.3	88
12	257	6.4	4	77.4	48
13	166	4.2	1	54.6	13
14	158	3.9	2	54.6	28
15	121	3.0	0	45.0	0
16	67	1.7	1	27.2	16
17	151	3.7	1	63.0	17
18	79	2.0	1	36.0	18
19	68	1.7	0	32.3	0
20	34	0.9	0	18.0	0
21	38	1.0	0	38.0	0
22	61	1.5	0	33.0	0
23	45	1.1	1	25.3	23
24	72	1.8	1	43.4	24
25	35	0.9	0	22.5	0
26	29	0.7	0	18.2	0
27	31	0.8	0	21.6	0
28	39	1.0	0	28.0	0
29	26	0.6	0	17.4	0
30	10,812	270.3	203	8,109.0	6,090

entail investigation of a wide range of plant capacities.

In order to save time, this problem was programmed for a computer* using a share assembler to enable it to be written in mnemonic. Any number of data can thus be used by dividing them into groups, not to exceed 6,000, which is the number of locations of core storage reserved. A program modification was used to enable the 40-year record to be analyzed as a unit. A test word was placed at the end of each block of data to inform the machine of the end of data. By placing a test word at the end of each year's data, the data were also analyzed on a yearly basis.

The data cards were punched in integer form with about 20 days of record per card. Calculations were thus carried out in fixed-point form, which is a little faster than floating-point form. The total number of cards came to about 750. Tape read-in and write-out were used for three reasons: [1] it is 25 times as fast as cards and an on-line printer; [2] the bulk of machine time was used in read-in and write-out rather than calculating; and [3] a very large amount of data was involved.

The program as written included only the number of days the yield was available. Table 1 illustrates the data and how Points *A* and *B* were obtained for the curves in Fig. 1. Average full yields for the average year and driest year, respectively, were calculated by dividing the totals of the "Full Yield" columns in Table 1 for these years by the number of days in the year. This gave 25.2 mgd for the average year and 20.1 mgd for the driest year. For a general case both the amount of reservoir storage and the filtration plant capacity may be varied in the program.

Curves of the type in Fig. 1 should be of assistance in deciding on the feasibility of the operational procedure and what would probably be the most economical size for the filtration plant. There still remain numerous other conditions and variables that must be considered, however, such as the modification of these curves for plants of various sizes not operating at full capacity and using all available runoff at all times. Because of the complexity of the interconnected system of which the Whitney reservoir is only a part, such a modification would be extremely difficult.

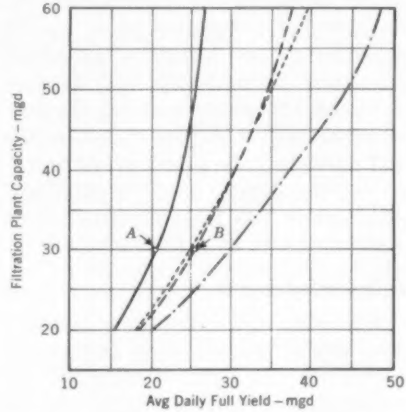

Fig. 1. Relationship of Plant Capacity to Average Daily Full Yield, 1918–57

These curves are based on calculations from data such as that presented in Table 1, which gave the locations of Points A *and* B. *Solid curve, 1930 (driest year); dashed, 40-year average; dotted, 1953 (wettest year); dashed and dotted, 1934 (evenly distributed wet year).*

* Model 704; a product of International Business Machines, Inc., New York, N.Y.

The curves also show the effect of a more even distribution of runoff when the curve for 1953, the wettest year, is compared with 1934, which ranked seventh.

Further investigations should include a study of wet and dry periods of years and the effect of extensive watershed development.

It was not the purpose of this study to arrive at any decision as to the feasibility of a filtration plant, but to demonstrate the usefulness of digital computers in analyzing long periods of record. It is estimated that the time saved in this study was between 800 and 1,000 hr. Programing and transferring the data to cards required approximately 120 hr, and actual computer time was only 14 min.

Acknowledgment

All work with the computer was done at the Massachusetts Institute of Technology Computation Center, Cambridge, Mass.

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Water Supplies in the Central and Western Canadian North

J. W. Grainge

A contribution to the Journal by J. W. Grainge, Dist. Engr., Public Health Eng. Div., Dept. of National Health & Welfare, Edmonton, Alta.

SETTLEMENT and development of the Canadian North are accelerating at a rate that is staggering to those who in past years have known it principally as a vast native hunting and fishing ground. The reasons for this rapid development are many and varied, the more important being extensive oil exploration, continuing development of hard-rock mines, construction of many residential and day schools, hospitals, government offices and residences, construction of the Distant Early Warning (DEW) Line, an extensive network of all-weather roads, and northern extensions to two railways in Alberta and British Columbia. Several new towns are expected to be located along these roads and railways.

Rapid development of many communities has resulted in many problems of water supply, sewage disposal, and sanitation. In the Yukon and Northwest Territories there are now about a dozen communities served by water and sewerage systems, not including the DEW Line stations. Within the next 5 years there will be at least five more communities with water and sewerage systems, two of which will be located along the Arctic coast.

The intent of the author is to provide information on northern water supplies; by this report of past successes, failures, and the conclusions to be drawn from them, others may be informed of the mistakes that can be made in these operations. The area under consideration is the Yukon Territory and central and western Northwest Territories, including the southern Arctic islands (Fig. 1).

The Mackenzie River flows through the center of a narrow northern extension of the Great Central Plains of North America. This rich petroleum belt consists of river valleys known as the Mackenzie Lowlands and the high Peel Plateau. The soil is generally composed of unconsolidated glacial and alluvial deposits of comparatively recent origin. It is bordered by two mineral-rich rocky areas, the Cordilleras in the Yukon Territory and Alaska on the west, and the Canadian Shield on the east.

The islands in the Arctic Ocean are in the geographical plain known as the Arctic Archipelago. These islands have an undulating to flat topography and are covered with sedimentary rock, principally limestone. Wildcat wells are being drilled with expectations of finding rich oil pools underlying these islands.

Most of the settlements were originally trading posts surrounded by a

few log cabins or native tents. The traders were followed by Anglican and Roman Catholic missionaries, many of whom built hospitals and schools with government assistance. Since the last war, construction of government schools, airports, and residences has greatly added to these communities.

In this haphazard way, settlements have sprung up at locations which, although suitable for log cabins, may present difficult problems when general water and sewerage systems and large buildings are contemplated. Several of these settlements are on silt deltas that are subject to considerable frost heaving. In such geological formations wells are generally unsatisfactory.

Climate

The climatic zones (*1*) of the Canadian North are sub-Arctic and Arctic. The usual distinction is that the mean temperature of the warmest month in the Arctic is lower than 50°F. The line of separation corresponds roughly with the tree line. It runs generally in a northwest direction from the southern shore of Hudson's Bay, continues north of Great Slave Lake along the northern shore of Great Bear Lake, and intersects the Alaska–Yukon border less than 100 mi from the Arctic Ocean. Both the midsummer and midwinter climates of the sub-Arctic are almost as pleasant as that in such population centers of central Canada as Saskatoon, which is only 250 mi north of the 49th parallel, although the winters are longer in the northern region.

The climate of the Arctic regions is much more severe, and midsummer and midwinter temperatures average about 20°F lower than those in the population centers of central Canada.

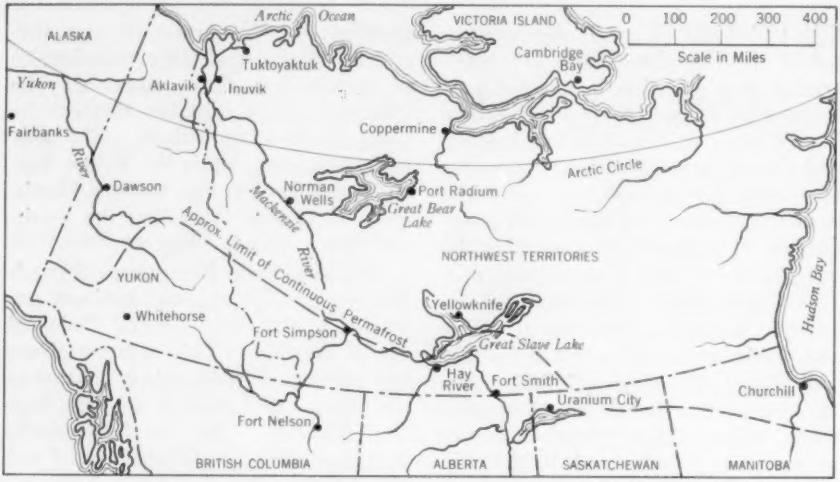

Fig. 1. Map of Central and Western Canadian North

Shown are all the settlements whose special water supply problems are discussed in this article.

The summers are short, there being only 6 weeks of summer in the central Arctic and 4 months in the western Canadian Arctic.

Wells

In bygone years the Indians, traders, and placer gold miners usually traveled by boats in summer. The Eskimos along the Arctic coast usually congregated at the mouths of the rivers to fish. As a result, the settlements invariably grew up on the shores of lakes or rivers where adequate quantities of fresh though often turbid water were always available. Wells were dug as deep as possible by hand, and if suitable water could be so obtained, it was used. Otherwise the surface water was used, regardless of its condition.

In recent years well-drilling rigs have been brought into the North and many successful wells have been drilled. At the motels at Mile Posts 1,033 and 1,095 on the Alaska Highway 100 mi from the Alaskan border, artesian wells have been developed by drilling 160 ft, approximately 90 ft of which is through permafrost. It has been found that these wells can be kept flowing either with an electric heating cable or by permitting continuous flow.

At Fort Smith at a depth of 170 ft is a limestone stratum in which large-capacity wells could be developed. This has not been done because the water contains 700 ppm hardness.

At Fort Simpson there are 50-ft wells but the nitrate and chloride contents are high. Infiltration of septic tank effluent is suspected of being partially responsible.

A 32-ft, permafrost-free well has been drilled below Marion Lake at Fort Rae, which is in a permafrost area, but the water is very hard.

At Hay River, large-capacity test wells 15 ft deep have been drilled on the lake shore. Infiltration wells will be used when a water system is built.

Rivers

The water in most rivers is turbid in summer and clear when the rivers are ice-covered. Generally the water in the rivers is uncolored, but Hay River is an exception.

There is now only one water system where turbid river water is used. This is at Fort Smith on the Slave River. The water receives standard treatment—alum feed, mixing, settling, and filtration.

A water system for the settlement of Fort Simpson, an island at the confluence of the Liard and Mackenzie, is now being planned. Here again, standard water treatment will be provided.

At Inuvik, a new mainland community 35 mi east of the delta town of Aklavik, a lake will be used as a storage reservoir for supplying the community when the river is turbid. River water will be lifted into Hidden Lake two or three times a year when the water in the river is clear. At such times the water will be fed into the distribution mains, the only treatment being chlorination. Treatment of water from the lake will consist of microstraining and chlorination. This unconventional system was designed as a result of the difficulties of constructing a permanent water intake and conventional water treatment plant where there is unstable soil and deep permafrost. It is estimated that the reduction in capital cost is approximately $500,000 and that there will be substantial savings in operational costs.

In order to prevent contamination of the rivers, because natives and freighting boats depend on the river for drinking water, disposal of sewage to the river is not advisable. The amount of available land in the North makes lagooning of sewage the most suitable method of disposal.

Lake Water

Although the mean annual precipitation throughout most of the Arctic and sub-Arctic is less than 10 in., there is an abundance of lakes. There are two reasons for this; lake beds are in many cases watertight, and the surface evaporation is low. The rate of evaporation is low because the water is cold. The time of evaporation is short because the lakes are ice-covered for the larger portion of the year.

Fig. 2. Above-Ground Summer Distribution System

Water is taken from a delta lake, which is flooded during the spring breakup of the MacKenzie River. It is chlorinated and filtered to remove small water bugs and particles. The ditch is used to convey wash wastes (not from toilets). Silt gradually slumps into the ditch, which must be reexcavated each year.

The lake beds are watertight because much of the central Arctic and sub-Arctic is covered with Pre-Cambrian rock, and the lakes occupy the myriad glaciated depressions. There are few fissures in this rock and the lake beds are thus water sealed. Many other lake beds are made watertight by the permafrost below them. There is no permafrost immediately adjacent to the water in the lakes, because of the moderating influence of the water, but permafrost may exist below this adjacent zone.

There is variation in the depth of both the upper and lower surfaces of the permafrost underlying lakes in areas of deep, continuous permafrost. The unfrozen-ground layer below very large lakes is thicker and the permafrost layer is thinner owing to the reduced heat loss to the water surface. In some cases this effect could lead to a complete break in the permafrost, and the lake could be spring fed (*2*). The survey crew, looking for a better site (recently named Inuvik) for Aklavik, in 1954 found several lakes that contained water markedly unlike that in other nearby lakes. It was thought that these might have been spring fed, but in no case was this ever investigated thoroughly. Twin lakes adjacent to the river, which are 4–6 ft deep, were investigated, and it was found that permafrost existed 2–3 ft below the bottom of the lake. Permafrost is close to the ground surface in the neck of land between the lake and river.

An interesting phenomenon observed in deep lakes in winter is the stratification of the water. This is caused by variation in density with temperature. Water's greatest density is at 39°F. Advantage of this fact has been taken by the company town of

Eldorado Mining and Refining Ltd. near Uranium City, Sask. The water intake in Beaverlodge Lake is located 600 ft off shore where the water is 45 ft deep. The temperature of this water varies between 36 and 38°F when the lake is frozen over and the water near the shore is 32°F.

One of the worst problems encountered in shallow-lake water sources is the increase of the concentration of salts in the water as the winter season progresses. In a typical lake on the south shore of Victoria Island in the Arctic Ocean, the water depth might be 10 ft and the taste of salts unnoticeable in summer, but in midwinter when the ice is about 6 ft thick, the water may be unpalatable.

Mackenzie River Delta Lakes

Lakes in the Mackenzie River Delta deserve special mention. The delta is not fan-shaped like the average one, but is confined between two hard ridges, the Richardson Mountains on the west and the Caribou Hills on the east, and as a result is elongated, being 40 mi wide by 140 mi long.

During May of each year the delta is flooded by the melting ice and snow from the southern parts of its considerable drainage area, which at one point is only 220 mi from the 49th parallel.

As in all north-flowing rivers, the ice in the Mackenzie River disintegrates not so much by melting as by the lifting action of the rising water. The water in the river may rise as much as 30–40 ft and the churning of water and ice is a phenomenal sight.

When this tremendous volume of water reaches the 5,000 sq mi of delta it spreads out over the wide expanse. Water in the delta channels rises 15–20 ft, depending on the location, particular year, and the location of the ice jams. The thousands of elongated lakes that formerly were channels in the delta are filled with flood water, and the delta becomes a vast reservoir until the ice jams move out into the Arctic Ocean and the water level in the delta channels falls.

The quality of the water in these delta lakes is generally superior to that of the mainland lakes. There is little algal life in the water, and the chemical quality throughout the year varies very little from the Mackenzie River at the time of breakup (Fig. 2). The iron content is negligible compared to that of the nearby mainland lakes, which is 1–60 ppm. The color is usually less than 15 (platinum–cobalt scale), and the turbidity, though over 100 ppm at the flood stage, drops to a negligible concentration within a few days. Aerial photos point up the contrast between the clear, blue water in the delta lakes, the turbid, gray water in the channels and the iron-containing, brown water of many mainland lakes.

TABLE 1

Chloride Content of Tukoyaktuk Bay Water in May

Depth of Sample* *ft*	Chloride Content *ppm*
6	129
10	238
13	303
16	377
19	499

* Thickness of ice was 5 ft; total depth of water and ice at point of sampling was 21 ft.

Water Supplies From the Ocean

In southern latitudes the cost of producing potable water from ocean water is high, but in the Arctic Ocean nature itself aids in making fresh water available. One favorable phenomenon is

the downward displacement of the heavier salt water by fresh water. After the Arctic Ocean begins to freeze over in late October, the fresh water of the Mackenzie River spreads out over the ocean water at the river mouth and gradually displaces the ocean water downwards, with very little mixing of the two. At Tuktoyaktuk, 18 mi east of the eastern delta channel, there is only 1–2 in. fresh water below the ice surface in October; by late December there is a sufficiently large layer of fresh water to be piped from the ocean; by breakup time in late May, the water is fresh for a considerable depth. On May 27, 1957, 1–2 weeks before breakup, samples of water were taken from several depths in the bay, and results of these observations are given in Table 1.

Drinking water is obtained from the ocean in winter, and sewage is disposed of on top of the ocean ice with little or no danger of contamination of the water.

Water Supplies From Ice

A second phenomenon that produces fresh water, though in very small quantities, is the extrusion of salts in the formation of ice crystals. Even after the ice has formed, the salt may be gradually extruded if the ice temperature rises. Advantage of this fact is taken by travelers in winter when they are near or on the ocean ice, and even in summer wherever drift ice floats near the shore. When the ocean ice breaks up in summer, clusters of the drift ice float about with jagged edges protruding above the surface of the

Fig. 3. Frost-Heaving Phenomenon

The ground around the circular patch of snow is fractured. This is on the mainland 15 mi south of Aklavik.

water. After freezeup in the fall, this ice may be melted and used for drinking water.

The ice that is pushed up in the pressure ridges on the ocean becomes salt-free later in the winter. In an actual test in May of 1957, for example, the water obtained from ice from a pressure ridge contained only 169 ppm chlorides, whereas that from the ice on the ocean surface 10 ft away contained 2,830 ppm.

In many places ice on lakes or rivers is the only suitable source of water. When large quantities of water are needed, as at some of the DEW Line construction camps, a variety of methods of harvesting this ice are used. Often long grooves are cut in the ice 18–24 in. deep with beaver-tail chain saws. The ice between the cuts is then chopped and loaded on trucks with mechanical loaders. Sometimes a tractor-drawn ripper is used to break up the ice on the lake surface.

Permafrost

Before consideration of the problems of construction and maintenance of pipelines and buildings, a brief discussion of the definition of permafrost is advisable. This is a very complex matter, and many technical volumes have been written on the subject in both Alaska and Russia. In Canada, the Division of Building Research of the National Research Council has devoted considerable time to this study since World War II and has published several reports. This discussion will cover only the barest essentials for an understanding of the information to follow.

Permafrost is ground, the temperature of which is perennially below the freezing point of water. Continuous permafrost covers almost all of the Northwest Territories, the northern half of the Yukon and the northern fringes of Manitoba and Quebec (*1*).

The upper surface of the permafrost zone is bounded by a layer of ground, which freezes and thaws according to the season and is generally referred to as the "active layer." The depth of this layer varies with the soil cover and the mean annual air temperature. Where there is moss cover, the active layer is usually only a few inches thick in the Arctic and as thick as 2 ft in the sub-Arctic. Where the ground is bare it is usually 4–10 ft thick or more, depending on soil condition. The shallowness of the active layer where there is moss cover presents an interesting point. The moss dissipates heat by the evaporation of water from its surface (*4*). This feature will be discussed further.

The lower surface of the permafrost zone varies considerably. There is very little information on the subject in the Arctic regions of Canada. Mining and oil companies at various locations report the following permafrost depths: Yellowknife, 150 ft; Port Radium, 325 ft; Norman Wells, 100 ft near the river (probably deeper farther inland); Cornwallis Island, more than 1,000 ft (*8*).

Permafrost is discontinuous along the southern fringes of the sub-Arctic. Even in what is considered to be the temperate zone there are islands of permafrost in the swamplands. In the more northerly areas, permafrost-free areas are found below cultivated soil and in open country.

Sections of unfrozen ground in permafrost are known as "taliks." These may exist as islands of unfrozen soil between the permafrost and the active layer or as complete breaks through to the unfrozen ground below the permafrost. Taliks have been used as water sources in Russia (*2*). The

water is sometimes under pressure in the taliks and many unusual occurrences have resulted from this phenomenon. In Russia the permafrost beneath one building was melted by the heat from the building, and water from a talik welled up through the floor and out the windows, forming frozen waterfalls below the sills. It is speculated that the frost mounds, called "pingos," along the western Canadian Arctic Coast result from this phenomenon. The largest of these is located at Tuktoyaktuk and is 100 ft high.

Frost heaving (Fig. 3) is a problem to builders as far south as the northern United States, but the effects are much more pronounced in permafrost areas. As in southern areas, however, frost heaving happens only in permeable silt, silt-clay or silt-sand soils (*3*). The presence of organic material complicates the problem. The distribution of this frost-heaving soil is unfortunately widespread in some of the more populated parts of the Northwest Territories.

Frost heaving takes place because the formation of ice crystals in the voids of a saturated soil at the interface where the water is being frozen as a result of heat transfer to the ground surface. Water travels to this interface by capillary action, feeding the growing ice crystals. The ice layer continues to grow thicker at the rate determined by the permeability of the soil as long as the frost line remains constant. The strength of the freezing crystals is tremendous and will raise any building or any soil burden above it. These ice crystals often form very fine layers less than $\frac{1}{8}$ in. thick and separated by soil layers. Under favorable conditions the ice lenses are thicker than the soil layers between them; some, called massive ice lenses, grow to thicknesses of a foot or more. At Aklavik this ice has sometimes been used for drinking. The excavations from which the ice is taken are used as natural refrigerators.

Massive ice lenses are common in the silt soils of the Mackenzie River Delta. The subsoil at Aklavik is subject to greater frost heaving than that of any other soil known to the author. One soil core removed from a 10-ft deep test hole by the field team of the National Research Council in 1953 was melted down and found to contain only a foot-high column of dry soil (*5*).

Building Construction in Permafrost Regions

The construction of buildings in regions of permafrost presents different problems in each area, which must be understood by the design engineers and builders intending to do work in northern Canada. Many buildings have been constructed in permafrost areas without any consideration of these problems, and some most peculiar effects have resulted. On the other hand, some builders, believing they had permafrost problems, have gone to considerable extra expense in the construction of buildings on ground where frost heaving was not a problem.

The main problem is the extent to which the ground heaves differentially below various parts of the building. In the late 1940's a two-story, frame hotel about 40 ft by 100 ft was constructed on mud sills in the settlement of Hay River, which is on a delta island at the mouth of Hay River. Later short piles were added. There was differential heaving and settling of the building, with grades up to 5 per cent, and the floor became generally dish-shaped. Problems stemming from the frost-heaving characteristics of the silt and organic soil at Aklavik

were responsible for a decision to construct a new settlement, recently named Inuvik, on a gravel deposit on the mainland on the eastern side of the delta.

Where the foundation soil is gravel, coarse to medium sand, or tight clay, the soil is stable and is not subject to frost heaving. No unusual precautions need be taken when designing foundations for buildings on such formations (*3*).

If the foundation soil is silt, silty clay, silt and sand or organic materials, special precautions must be taken to prevent damage to the buildings by differential frost heaving and settlement. This subject has been dealt with in considerable detail in many technical papers (*2, 3*). Briefly summarized, the most important of their recommendations are as follows:

1. The building should rest on piles anchored in the permafrost to a depth twice that of the active layer.
2. In addition to placing the building on piles, one may also preserve the thermal regions of the present ground by adding insulating material over the soil.
3. An open air space below the floor protects the soil below the buildings from the heat from the inside.
4. The unstable soil should be excavated and replaced with gravel or sand if piles are not used.
5. Small, low-cost buildings should only be built on mud sills.

The way to construct a large water treatment or other kind of building, with an underground water reservoir, is exemplified in the largest building in Aklavik, which is located on an island in the delta of the Mackenzie River. This building is a residential school, 140 ft by 35 ft, two stories high, and has a basement ceiling 9 ft high. The reinforced-concrete basement walls are 14 ft high and flare out at the base. The bottom of the wall footing extends 12 ft below the ground surface and 5 ft below the concrete floor. Both the 1-ft thick concrete footings and the 1-ft thick basement floor rest on gravel, which was placed there prior to construction. The basement floor is drained to a sump, and in spring there is a small discharge of ground water through one or two cracks in the floor.

When last examined in 1958, there were no indications of either settlement or frost heaving below the wall footings. There was a small amount of settlement below the basement floor on the south side of the building, but no sign of frost heaving.

Water and Sewer Pipelines

Pipelines may be laid above ground or buried, there being several alternative methods of heating and insulating each of these systems. These will be discussed briefly.

"Utilidors" are made of insulated boxing which contains the water or sewer pipes and usually steam and steam condensate pipes. They may be constructed above ground (Fig. 4), in which case they are usually weatherproofed and insulated, or below ground, in which case provision must be made for drainage. Some, such as those at military bases near Fairbanks, Alaska, are made of concrete and are large enough for workmen to enter. Canadian experience is limited to simpler construction except in southern latitudes.

The hard-rock–mining towns in the Northwest Territories use these systems. These settlements are located on solid rock and are compactly laid out. The above-ground utilidors contain pipes for water, sewage, steam,

and steam condensate for heat and service for all buildings.

Utilidors also have been used at a few company towns located on sedimentary deposits, the most interesting of which is the oil refinery settlement of Norman Wells (Fig. 4). In this settlement steam heat and drinking water are also supplied to government buildings almost a mile from the heating plant. This system was constructed at the time of the development of the Canol Pipeline extending from Norman Wells to Whitehorse by the US Army.

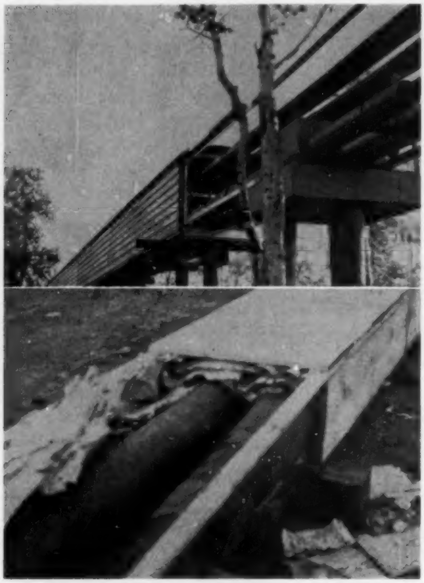

Fig. 4. Utilidors

The upper photograph is of a utilidor under construction at Inuvik. Two top pipes (welded steel) contain hot water; bottom outer pipes (asbestos-cement) are for water and sewerage; and the bottom, middle pipe is for replenishing Hidden Lake with water from the Mackenzie River in winter, when the river is clear. The utilidor at Norman Wells (lower photograph) rests on wood blocks on earth mounds.

The soil at Norman Wells is mainly silt, containing fine sand and scattered lenses of gravel and clay, and is subject to considerable frost heaving (*6*). It is not surprising, therefore, that the underground utilidors that were first used were found too difficult to maintain. These metal utilidors were laid with insulation at shallow depths.

They were replaced by wood frame utilidors, weatherproofed, insulated with asbestos paper, and set on wood piles spaced approximately 8 ft apart. These piles were reportedly driven into the solid ground but were not anchored securely in the permafrost, and it was found that heaving and settlement were a problem.

During the summer of 1954 the utilidors were laid on small wood blocks on earth mounds. It was found that earth mounds are easier to maintain after differential settlement than are piles and that additional insulation is provided by the drifting snow cover in winter.

Long, buried pipelines are successfully used in Fort Smith, Whitehorse, Yellowknife, and Dawson. The first two places are in regions of discontinuous permafrost, and there is no permafrost in the particular places where the pipes are buried.

At Fort Smith the pipes in the distribution system were originally laid in 1950 with 9-ft soil cover, and in winter the householders at the ends of the lines kept taps running slowly for 24 hr a day. These pipes have since been relaid with 11–12-ft soil cover and there have been no problems with freezing. The bleeding of the lines is not now required.

At Yellowknife, where the permafrost extends to depths of 150 ft, a recirculating, dual-main distribution system was laid in 1949. There are two separate systems of mains and service connections. Heated water is supplied to the homes by a high-pressure system. Water flows from one system to the other through an orifice connection in the basement of each building and returns to the water plant through the second system.

The single-main recirculation system designed by W. Page of the Arctic Health Research Center, Anchorage, and constructed at Fairbanks, Alaska, has probably outdated the dual-main recirculation system used for large distribution systems.

At Dawson the wood stave mains are only 2 ft underground and some service pipes are less than a foot deep. In winter water is heated to 40°F. Sufficient water flow to prevent freezing is maintained by bleeding in each house and at the dead ends of the mains. Hydrants are wind-tight and heated with 100-w heaters.

One of the most inexpensive and successful methods of laying a pipeline is to lay the pipe on the ground surface and cover it with 3 ft of moss. The moss provides good insulation by entrapping large amounts of air and supporting a heavy blanket of snow. In practice, moss has been demonstrated to be almost valueless as insulation around buried pipes, however, because it becomes water-saturated when buried (*4*).

There are many other methods of insulating pipe. Some pipe is manufactured with insulation already on it. The amount of insulation around a pipe must be calculated according to the temperature of the water, the expected air temperatures, and the rate of pumping. At the Anglican mission school and hospital in Aklavik, water is piped through about 600–700 ft of uninsulated $2\frac{1}{2}$-in. iron pipe in the coldest of weather. The water is heated by pumping it first through a homemade wood-burning, jacket hot-water heater.

At the sites on the DEW Line, sewage at about 65° F is disposed of by pumping it out through 500–600 ft of uninsulated, 3-in. aluminum pipe. Trouble was experienced only at those sites where complete drainage of the line after each pumping was not possible owing to dips in the line.

Water Supply Lines

A few water supply lines up the high (75–175 ft) river banks of the Mackenzie River or its tributaries are particular problems. Many of these banks are fairly steep, but when the moss and shrubs are stripped during construction the permafrost melts and the saturated bank slides.

A similar effect may result when matted surface cover is removed, leaving this ground subject to surface water erosion. This occurred at Fort Nelson on the Alaska Highway, where in 15 years all the earth from around a 20-ft deep tank slid down the bank.

At Fort Smith this problem was encountered in 1951, the first year after construction of the water supply line. The engineer introduced 12-ft sections of rubber hose in the line at both the top and the bottom of the bank. The pipeline was laid in an insulated, waterproofed boxing which merely rested on the ground surface. In the 6 years since this alteration the bank has slipped sufficiently to extend the top rubber hose to its full length.

This supply line is being replaced with pipe of larger diameter and the

consulting engineer has proposed that manufactured insulated pipe be used. This is to be mounted on wood blocks and the permafrost raised by placing a foot or two of moss over the area.

Heating of Supply Lines

There are a number of alternative methods for prevention of freezing in supply lines. In many cases the lines are contained in utilidors and heated with the steam pipe supplying heat to the pumphouse. At Fort Smith, water is heated in a hot-water tank and discharged through the supply line to the intake well each time before pumping. The line is drained by the opening of a motorized valve each time the pumping stops.

For a proposed water supply line at Fort McPherson a different method was recommended. It was suggested that a hot-water heater and a tank with a thermostat be installed in the pumphouse. All water would be pumped through this to the supply line. The line but not the hot water tank would be drained by use of a motorized valve.

At some locations all water is heated to a few degrees above freezing in coils in portable, wooden boxes located at the pumphouse. These pipelines are also drained after pumping.

Conclusion

This paper has covered some of the problems of water supply peculiar to the central and western Canadian North—the sources of water, the heating and insulation of pipelines, and the construction of buildings and pipelines where there is permafrost on frost-susceptible soils. Various solutions to these problems have been briefly described, but, for the most part, discussion of the more technical, engineering aspects has been avoided in order that the larger picture might briefly be presented. It is hoped that others planning to do work of this kind in the North may benefit from the experiences recounted here.

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Water Supply System of Caracas, Venezuela

Joseph Sprecher

A contribution to the Journal by Joseph Sprecher, Mech. Engr., Pump Div., Sulzer Bros. Ltd., Engrs., New York and Winterthur, Switzerland.

IN August 1956 the commissioning ceremonies for one of the largest water supply systems in South America took place in Caracas, Venezuela. This was the final act in the execution of a $42,000,000 project.

The population of Caracas has almost tripled in the last 15 years and is now more than 1,100,000. The city's water supply, always something of a problem for the authorities, understandably proved unequal to this rapid increase in demand. Before 1956 Caracas was supplied by ground water and by a number of reservoirs filled by the runoff from the surrounding hills. An average of about 26,000 gpm is available from these sources. Caracas is located in a tropical region, however, where 6 months of heavy rains are followed by an equal period of drought. Caracas's water requirements were not unduly difficult to satisfy during the rainy season, but during the dry months there were quarters of the city which were without water for days on end. The unusually swift growth of the city naturally aggravated these conditions and relief measures soon became imperative.

Beginning in 1947 the National Institute for Sanitation had a number of water supply projects prepared. Nine projects were carefully studied and compared before the Rio Tuy–Mariposa project was selected.

Tuy–Mariposa System

The Rio Tuy–Mariposa system comprises a feeder station and four main pumping stations which operate in a series and raise the water a total of about 3,130 ft. The total input of the pumps in these stations is about 60,000 hp. The steel conduit is 18 mi long altogether. Figure 1 shows an outline of the system.

The Tuy River, which rises at an altitude of about 7,900 ft on the southern flank of the coastal cordilleras, is dammed above Santa Teresa, about 15 mi from Caracas. The water for Caracas flows first through three screen-protected intakes into a sluice chamber, and then through two concrete pipes, each $48\frac{1}{2}$ in. in diameter, and into the two suction chambers of the riverside feeder station. Four vertical mixed-flow pumping sets are installed in this station, two in each suction chamber. Each of the four units is designed to handle 12,150–13,730 gpm, against a head of 22–$19\frac{2}{3}$ ft, when running at 590 rpm. The pumping sets deliver the water through nonreturn valves into a supply conduit whose branches lead into three settling and desilting tanks. Each tank is 194 ft long, 59 ft wide at the top, and 10 ft deep. About 80 per cent of the entrained substances in the water are precipitated in these settling tanks. The desilted water is then enriched with carbonic acid in

three parallel tanks, each about 82 ft long. The water then flows into transverse clear-water tanks 197 ft long and 82 ft wide where chlorine is added to prevent the growth of algae. The four pumps of Station 1 draw the water from this tank.

Each of the four stations has four pumps. The sixteen pumps in the four stations, together with the feeder pumps, raise the water from an altitude of about 430 ft to an altitude of 3,560 ft, a total difference of about 3,130 ft. The delivery pipe to which all four stations are connected runs almost horizontally from its highest point for a distance of about 3.1 mi (Fig. 1) before opening into the Cor-

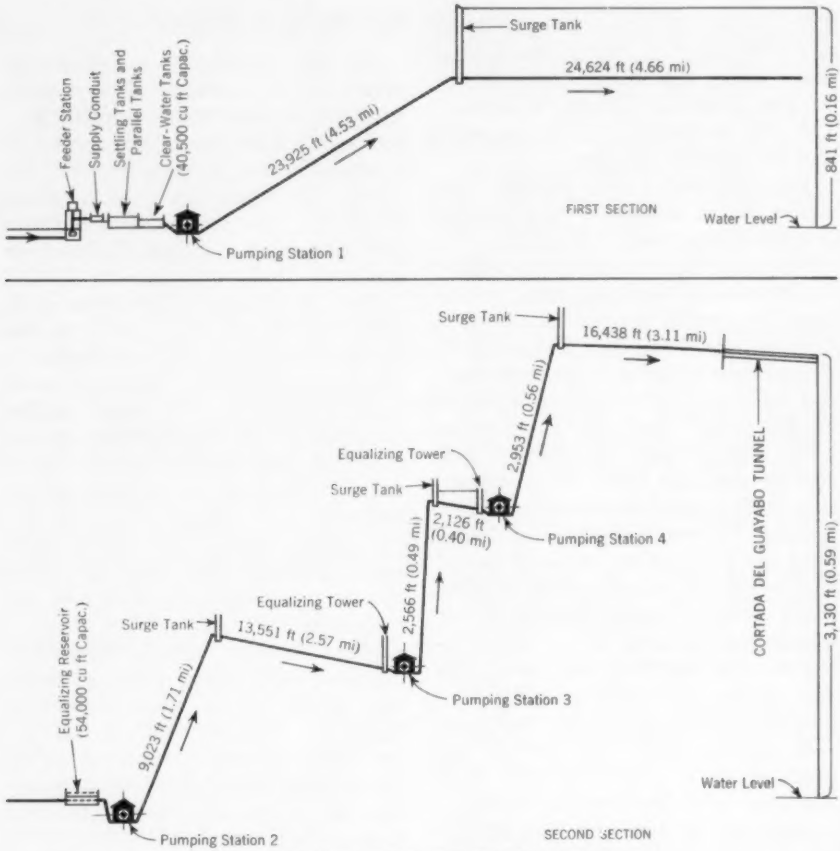


Fig. 1. Section of Rio Tuy–Mariposa System

This part of the system carries water through a steel conduit from an altitude of about 430 ft to a maximum altitude of about 3,560 ft, a rise of about 3,130 ft. Other distances refer to the length of section of the conduit. Total length of the conduit, from Pumping Station 1 to the Cortada del Guayabo Tunnel is about 18 mi.

tada del Guayabo Tunnel, which has an ID of 6 ft 7 in. and a total length of 5,050 ft. The water then flows down the bed of the Rio El Valle to the Mariposa Reservoir. This reservoir has a total capacity of about 335,000,000 cu ft and serves as a raw-water reservoir for the Caracas filtration plant about two-thirds of a mile farther down the valley. From the plant the filtered water goes into the city's distribution system.

Pumping Stations

An important problem in the Rio Tuy–Mariposa system is to raise about 46,000 gpm of pretreated water to an altitude of 3,560 ft, overcoming a static head of 3,130 ft and the attendant pressure losses it causes.

The division of the system into a feeder station and four main stations equipped with four pumping sets each, operating in parallel, was partly dictated by the maximum connected load of 3,500 hp per unit permitted by the Caracas electricity authorities. Four vertical, mixed-flow pumps are installed in the feeder station and are automatically controlled in accordance with the water level in the clear-water tanks. Each of these units is designed for the following performance:

Delivery—*gpm*	12,150–13,730
Total delivery head—*ft*	22–19$\frac{2}{3}$
Speed—*rpm*	590
Input—*hp*	100–99
Motor rating—*hp*	135

Figure 2 shows the test results and characteristics of one of these feeder pumps.

The positions of the pumping stations (Fig. 1) were fixed by the topographical conditions. The clear-water tanks are before Station 1; a surge tank and the equalizing reservoir are before Station 2; a surge tank and an equalizing tower 56 ft high and of 17,500 cu ft capacity are before Station 3; and a surge tank and an equalizing tower 46 ft high and of 14,000 cu ft capacity are before Station 4.

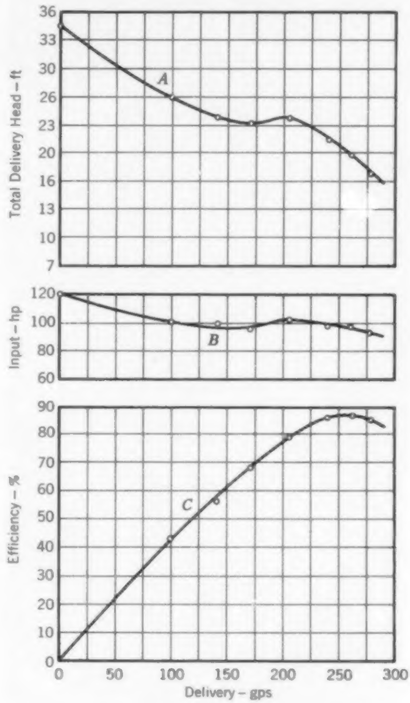


Fig. 2. Feeder Pump Performance

Curve A represents total delivery head, B represents input, and C represents efficiency of a feeder pump of the system.

The surge tanks (Fig. 3) consist of pipes of the same diameter as the pipeline, 3 ft 9 in. They stand upright and are surrounded by reinforced concrete supports. The equalizing towers of Stations 3 and 4 are cylindrical struc-

tures, each one 23 ft in diameter (Fig. 4).

The pumps in Station 1 deliver the water from the clear-water tanks into the equalizing reservoir. Stations 2, 3, and 4, however, form a single interconnected system, the pumps in each lower station delivering directly to the pumps in the station above. The two equalizing towers smooth out any fluctuations.

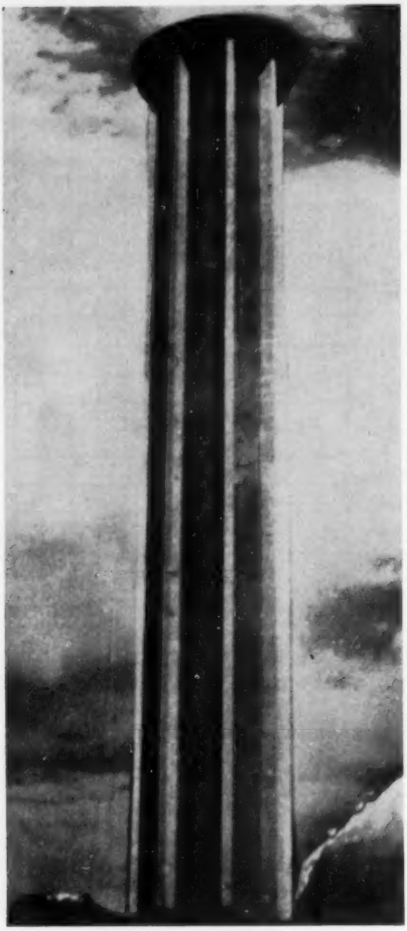


Fig. 3. Surge Tank

The first surge tank in the Rio Tuy–Mariposa system, placed about 4.5 mi along the conduit from Pumping Station 1 (Fig. 1).

Each of the four pumps in any given station is connected in series with a corresponding pump in each of the other stations. All sixteen main pumps are of identical design, a feature which is highly advantageous as far as the provision of spare parts is concerned. The pumps are two-stage, double-flow units in horizontal arrangement with diffusers and radial impellers and a common spiral casing (Fig. 5). Instead of stuffing boxes with soft packings metallic seals with inserted rings of the labyrinth type are used. The rotor runs in the water-cooled, ring-lubricated bearings and is flexibly coupled to the rotor of the three-phase motor.

The cooling water is taken from each main pump upstream from the first pressure stage and goes through an indicating flowmeter and a regulating valve to the cooling coils of the bearings. It is returned through a collector to the suction pipe, passing by way of a flow detector with an electric contact. Consequently, no cooling water is lost.

Each of the pumps in Stations 2, 3, and 4 is designed for the following performance:

Delivery—*gpm*	11,500
Total delivery head—*ft*	833
Speed—*rpm*	1,500
Input—*hp*	3,380
Motor rating—*hp*	3,800

When all of the series of pumps are in operation about 46,000 gpm are supplied to the city of Caracas.

Some characteristics of one of the main pumps are shown in Fig. 6.

The four pumps in Station 1 each absorbs 3,450 hp and delivers more water than the pumps installed in the other stations—12,150 gpm instead of 11,500 gpm.

The nominal performance data of the synchronous driving motors for the pumps are as follows:

Voltage—*v*	6,000–6,600
Current—*amp*	277–272
Speed—*rpm*	1,500–1,800
Frequency—*cycles*	50–60
Output at shaft—*kw*	2,800–3,020

All these motors are designed so that they can also be operated at 60 cycles per second. The system's frequency may later be raised from 50 to 60 cycles.

Operation

Station 1 normally transmits the operating control signals to Stations 2, 3, and 4. To illustrate this: it may be assumed that the whole plant is out of operation when the order to start up is received by telephone from the filtration plant at Caracas. One of the units in Station 1 is then automatically started by throwing the control switch. The 6-kv switch closes, the motor starts up, and at a certain speed the exciter is switched on. As soon as the motor is synchronized the discharge valve opens and the pump begins to deliver. The flow of water now produces a pressure difference in the pitot tube at the inlet to Station 2 and this provides the signal for the starting of the corresponding set in Station 2. The flow from Station 2 provides the signal for Station 3, and the flow from Station 3 provides it for Station 4.

The operator in Station 1 can follow the various automatic operations on a luminous indicator panel. As soon as the discharge valve is open in Station 4 the second and third series of pumps can be switched on as required.

The automatic equipment of the whole plant is so devised that the pumps in Stations 2, 3, and 4 start up only when the water level in the equalizing reservoir or in the equalizing towers is above a certain minimum.

It is also possible for a water-level signal transmitted from the equalizing reservoir to start up a pump in Station 1; the corresponding pumps in Stations 2, 3, and 4 then come into operation in succession as soon as a certain water level is reached in the equalizing reservoir. The single pumps in each station may also be started or stopped by hand with control switches. Finally, all the pumps may be started or stopped by manual remote control from main Station 1 by means of high-frequency currents superimposed on the high-voltage line.

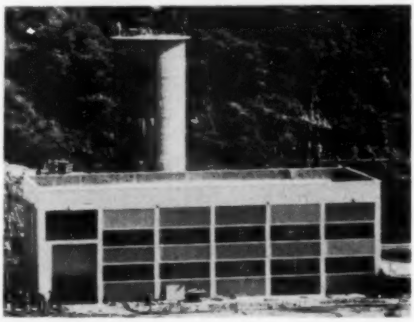


Fig. 4. Equalizing Tower

Pumping Station 3 and its equalizing tower. This tower, one of two in the part of the system shown by Fig. 1, is 23 ft in diameter.

The water levels in the equalizing reservoir and the towers, the switch positions in the high-voltage part of the transformer plants, and the working conditions of individual pumps (whether in operation or not) can also be transmitted to Station 1 and optically indicated there.

The usual procedure is to start one of the series of pumps and to wait until the whole series is in operation before starting the next, and possibly the third and fourth series, as required to meet the demand for raw water in the Mariposa Reservoir. The pumps are consequently started in sequence and at adequate intervals so that the electric mains cannot be overloaded. Time switches insure that the synchronous motors cannot be put into operation again until at least 20 min have elapsed since the previous startup.

Stopping, like starting, may be effected either automatically or by hand. If the pumps are to be automatically stopped the order to shut down is transmitted by the water-level detectors to the feeder pumps as soon as the level in the clear-water reservoir has reached the maximum height or the level in the suction well has dropped below 430 ft.

The main pumps are shut down either by hand—by operating the control switch in the station—or else by remote control, which may be used either to shut down a whole series of pumps or, if required, individual units only. The discharge valve is closed at the same time as the motor is stopped. The pump is therefore ready for the next start as soon as the minimum interval of 30 min has elapsed.

If the electric supply fails the discharge valves remain open. When the

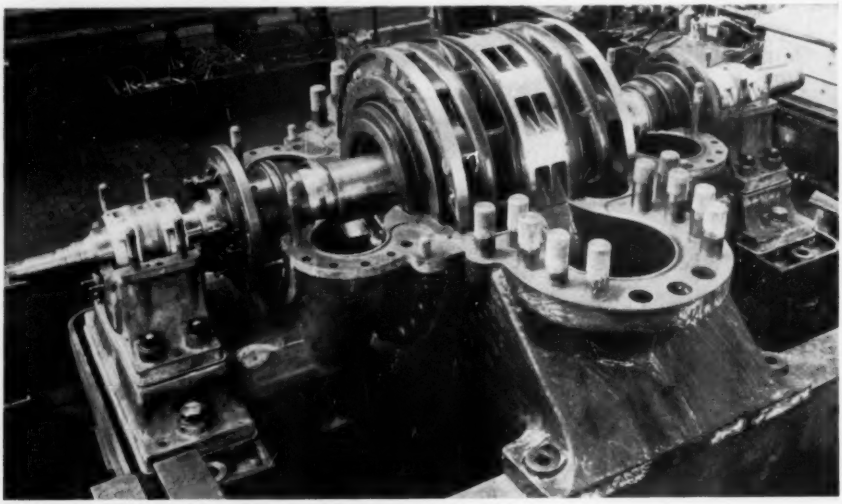


Fig. 5. Main Pump

A two-stage, double-flow main pump used in the system. Upper casing has been removed. This pump is designed to deliver 11,500 gpm.

supply is resumed these valves must first be closed before the main pumps can start again.

Power and Special Equipment

The three-phase, high-voltage line supplying electricity to the pumping stations is more than 13 mi long and is designed for operation at 115 kv. Stations 2, 3, and 4 each have two 7,500-kva transformers. Station 1 has two 8,200-kva transformers. The secondary voltage is 6,000 kva.

Safety is insured by interlocking systems and safety devices. In the feeder station two float valves regulate the water level in the suction well and prevent the pumps from running dry. Transmitters regulate pump operation in the feeder station in accordance with the level of the water in the clear-water tanks.

The main pumps in the stations have the following control equipment and special fittings:

1. Suction valves with limit contacts which allow current to be supplied to the driving motors only when these valves are open
2. Float switches with water-level transmitters which respond when the suction water level falls below the admissible minimum and thus protect the pumps against dry running
3. Water-level transmitters which insure that the maximum water level in the equalizing reservoir and towers is not exceeded
4. Discharge valves with limit contacts for remote-control operation and for automatic indication of the open or closed position; gears between these valves and their adjusting motors, protected by torque limiters which take action if any jamming occurs
5. Pump and motor bearings protected by double or single thermostats so that the pump in question is shut down if the maximum temperature of 80°C is ever exceeded.

The problem of pressure fluctuations has been treated in detail elsewhere (*1*). In the Rio Tuy–Mariposa plant the pipelines are fully protected against excessive pressure surges due to sudden power failures. In Stations 1 and 2, where the equalizing towers are several miles distant (Fig. 1), a considerable reserve of energy had to be created in order to prevent vacuum for-

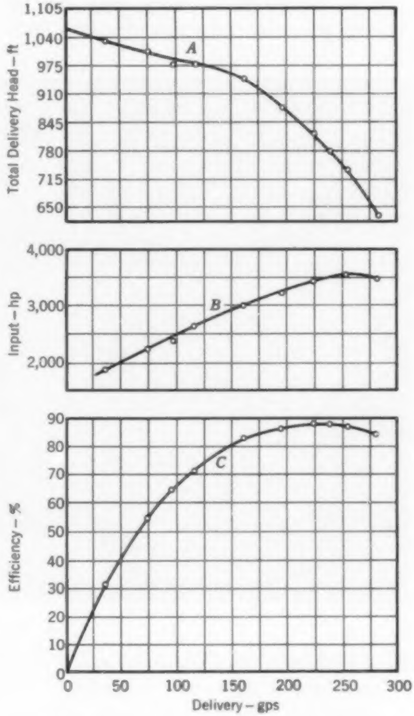


Fig. 6. Main Pump Performance

Curve A represents total delivery head, B represents input, and C represents efficiency of a main pump of the system.

mation in the discharge pipe. This energy reserve could only be provided in the form of large air vessels connected in parallel with the main pipe. These air vessels, which are located outside the pumphouse, also reduce to a harmless level the pressure shock following the first low-pressure wave.

The compressors supplying air to the vessels are controlled by two electrodes in accordance with the water level. If the water level nevertheless rises too high an alarm signal is given. The alarm signal of Station 2 is transmitted to Station 1.

In Stations 3 and 4 the length of the discharge pipe, which rises fairly steeply to the equalizing towers, is less than 3,000 ft, so that in these stations flywheels suffice to provide the energy reserve. These flywheels (Fig. 7) are solid, forged-steel discs slightly less than 5 ft in diameter and are keyed onto the ends of the motor shafts.

All the main and feeder pumps, as well as the principal fittings and water-level regulators, are equipped with safety and alarm devices which respond in the event of any abnormal conditions occurring. Thus, the main pumps are specially protected against hot running as a result of prolonged operation with the discharge valves closed. All the discharge valves are furnished with small bypass pipes for this purpose which allow a certain amount of water to flow off, insuring that the pumps are sufficiently cooled.

Two electrically driven vertical sump pumps are installed in Station 1 and

Fig. 7. Flywheel

Flywheels provide the energy reserve in Pumping Stations 3 and 4. This flywheel, in the main pump room of Station 3, is a solid, forged-steel disc, slightly less than 5 ft in diameter.

equipped with diaphragm switches. Two switches put them into operation in sequence and a third switch shuts them down. A fourth switch actuates an alarm if the maximum water level in the sump is ever exceeded.

The program of measurements carried out after shutdowns when the stations were commissioned yielded results which agreed very well with the calculated values for the equipment.

Reference

1. DUC, JEAN. The Calculation of Water Hammer in Pumping Plants. *Sulzer Tech. Rev.* (*Switz.*), No. 1 (1955), p. 22.

Water Losses From Evaporation

A preliminary report by the US Geological Survey estimates that more than 21,000,000 acre-ft of water is evaporated annually from the water surfaces of fresh-water streams, lakes, canals, ponds, and reservoirs in seventeen western states. This is more water than is now being used for all the cities and communities served by public supplies in the entire nation. The USGS data on evaporation are part of a study of the savings that might be made by reducing evaporation losses in areas where water is in short supply.

The following table shows annual evaporation losses for the seventeen western states and losses from the portions of stream channels and reservoirs that form the boundaries between states or nations. The latter are listed separately by river name and are not included in the state totals.

State or Boundary River	Annual Evaporation Losses *1,000 acre-ft*				State or Boundary River	Annual Evaporation Losses *1,000 acre-ft*			
	Large Lakes and Reservoirs*	Principal Rivers	Small Lakes, Reservoirs, and Streams	Totals		Large Lakes and Reservoirs*	Principal Rivers	Small Lakes, Reservoirs, and Streams	Totals
Washington	683	487	80	1,250	North Dakota	991	68	415	1,474
Columbia R.	173	372	—	545	Red R. (North)	46	7	—	53
Oregon	503	101	100	704	South Dakota	1,207	97	260	1,564
Snake R.	—	106	—	106	Minnesota R.	26	—	—	26
Idaho	968	248	84	1,300	Big Sioux R.	—	6	—	6
Montana	1,188	272	143	1,603	Missouri R.	37	369	—	406
Wyoming	431	71	85	587	Nebraska	210	392	429	1,031
Colorado	208	83	120	411	Kansas	192	268	217	677
Utah	390	161	63	614	Oklahoma	394	352	234	980
Nevada	538	3	30	571	Red R.	331	125	—	456
California	1,352	202	203	1,757	Texas	1,148	406	633	2,187
Colorado R.	1,135	230	—	1,365	Sabine R.	188	22	—	210
Arizona	94	209	119	422	Rio Grande R.	362	139	—	501
New Mexico	192	35	81	308					
					Totals	12,987	4,831	3,296	21,114

* More than 500 acres.

Evaluation of Supervisory and Automatic Control

Wellman E. Nusbaum

A paper presented on Apr. 18, 1958, at the Conference on Control, Instrumentation, and Automation in the Water Supply Field (cosponsored by the Missouri Section, AWWA), at the University of Missouri, Columbia, Mo., by Wellman E. Nusbaum, Black & Veatch, Cons. Engrs., Kansas City, Mo.

THE engineer's role in the application of automatic and supervisory control to water systems must extend well beyond the purely technical aspects of control design. Not only must a control system be designed to perform certain work functions, but the personnel who are to use it must be well trained. Certain controls may often be desirable but the utility management may be unable to supply related operation and maintenance requirements. The engineer in such instances must demonstrate his professional responsibility to public bodies by advising against the use of such controls. Anything labeled automatic appeals to the imagination, but management's dream may result in lasting dissatisfaction, needless investment, and excessive maintenance expense if the control is not patterned to the using organization or if the organization is not reshaped to fit the control. One way or the other, adjustments must be made.

The greatest desire and perhaps the greatest need for automatic equipment probably exist in systems where its use can least be warranted. Unfortunately, neither the desire nor the need alone can justify its installation. Plainly, then, to design properly, it is necessary to evaluate the application of controls in terms of the ability of the utility to use them efficiently.

Seldom is any particular salesmanship needed to promote the installation tion of an unattended station that would use automatic or remote supervisory controls. The acceptance of automatic controls by water utility management is surprisingly widespread. This is probably explained by the oversimplification of controls often found in enthusiastic advertising. In the past 10 years, people have read almost daily of push-button control of everything from warfare to housework. Missiles and dishwashers are remotely controlled or automatic—why not pumping stations? With satellites that were launched by controls circling the earth and collecting and transmitting information back to control stations, the remote control of an unattended pumping station appears, by comparison, a simple chore. No doubt it would be—for the same task force.

It is interesting to note that nearly any control scheme and instrumentation can now be made to operate at practically any distance. With this established, it is not hard to imagine how an ill-advised management and an

overenthusiastic control engineer might develop an unmanageable fantasy. Yet what might be an unmanageable system for one organization could be a very satisfactory one for another. What, then, is the difference? Again the difference lies in the specific situation which application of this equipment develops in a particular utility. It is not so much a question of whether it can be done; the engineer should be interested in whether it is the right place to do it.

Unwise Planning

A water utility organization might easily embark upon a project that is overextended into automatic and remote supervisory controls. In the beginning, the arguments for such equipment may have been quite strong—higher quality, higher production rates, and lower operating cost. That process instrumentation produces a higher-quality product is well established. Management, in most other industries, could expect an increase in production rates; water utility management is unusual in this respect because it cannot establish its production rate; the proper rate is determined solely by the demand. Water utility management may expect to reduce operating cost; generally, it envisions a reduction in the number of operators.

The original organization of this hypothetical system used manual labor. This labor was local and was in the full-time employ of the water department, but the new plant may use indirect labor. Indirect labor refers to the highly-skilled, nonproductive technician required to maintain the equipment.

This indirect labor may be neither an employee of the water department nor even available locally, in the smaller communities, in which case it becomes necessary to go outside of the department or the community for maintenance. This brings up a real problem. Who will maintain the equipment? Generally, local or plant electricians have had little, if any, technical training and are incapable of immediately assuming the responsibility of maintaining intricate controls.

The original system permits the repair or replacement of an individual and exclusive piece of equipment without affecting the operation of all other components of the system, whereas the new plant may be affected by faulty components, which are common and can disrupt the operation of the complete system. Solutions to this problem are simplicity of control schemes and the employment of a thoroughly-trained and experienced technician to carry out a sound preventive maintenance program.

The original system was comparatively easy, during construction, to check out and place into operation, whereas the new plant will undergo a long adjustment period. There will be a long period of testing the equipment under various operating conditions, locating and correcting manufacturing and installation errors, making changes, adjustments, and time settings in order to bring the complete system up to an acceptable level of performance. This may take weeks or months, depending upon the complexity of the controls, the skill of the workmen, and the quality and condition of the equipment. It is this stage of construction that has so often been overlooked. During this time, management may become impatient and overcritical; operators may become annoyed and uncooperative; the contractor is anxious to terminate the work; bills for equipment become due;

and the public expects the most in performance. This is, however, a period requiring the maximum understanding and patience of management, the fullest cooperation of operators, and the highest level of ingenuity on the part of the engineers and technicians.

It would be fine if a special task force (a crew of technicians trained in control and instrumentation) could place the station in operation and then be followed by operators especially trained to operate and maintain the equipment. This may be done, to a certain extent, by training operators and hiring a plant maintenance man who has had broad experience with instrument mechanics and electrical circuitry. Such a man should be fitted into the project during the early stages of construction. He should study the equipment, prepare preventative maintenance schedules, arrange for spare parts, acquire test and maintenance equipment, and learn as much as possible from technical representatives of the equipment suppliers.

The original system was characterized by the use of conventional, standard equipment; the new plant, however, will consist of varying amounts of special equipment, some of which may be custom designed. Supervisory-control equipment for remote instrumentation and control of a particular system is of necessity custom built. In this respect, the utility faces a compound problem—that is, the probability of obsolescence and shorter service life. Because of the rapid technological changes and because many supervisory-control suppliers are in their infancy, it is difficult to know whether replacement parts will be available in even 5–10 years, whereas 30–40 years is normal service life of a system. Equipment suppliers generally purchase the component parts from other manufacturers and may have little control over the discontinuation of their manufacture. The service life of the components may be uncertain, depending upon their tolerances, operating characteristics, and applications. Competition in sales of the equipment may influence the quality of components, which may be calculated to last only through the warranty period. The greatest protection in this regard seems to be to select equipment from a well established supplier who has satisfactorily supplied similar installations. This method of selection, unfortunately, places a hardship on the newer suppliers.

Whereas the original system was characterized by flexibility of operation, the operation of an automatic or remote-controlled station is quite inflexible. Once equipment controls are installed to perform certain operations under certain conditions, the slightest change in the method of operation may require major modifications of the control equipment. For example, to change an installation component designed to open and close a valve by a signal sent over a telephone interconnection leased to the utility to permit throttling control of the valve could require major modifications in the supervisory control's dispatching and outlying stations and in the valve-operating appurtenances.

Thus when one converts from attended stations to automatic or remote-controlled stations, one leaves one epoch of operation and enters another. This progression requires a considerable change in management's thinking and in the attending personnel requirements. Its effects on attitudes and

behavior may be even more influential than the technical changes. Conversion from steam-powered stations, for example, to remotely controlled, electric-powered stations, while maintaining the former operating personnel, may be difficult and trying. The new jobs may result in less physical fatigue but increased tension and mental effort. The operators will then be confronted with new demands that require additional training and education. Jobs that originally called for manual skills and physical labor will call for mental skill and judgment. The operators will also become coordinators and technicians, compelled to watch, endure tension and boredom, and intervene with immediate manual correction if anything goes wrong. Such operators must become analytical, develop the ability to conceptualize (rather than see) a situation, remain alert, and deduce quickly what must be done. To do this the operator should have the ability to learn the complete system, its equipment and controls. Management must recognize that these new job requirements are different; it must assume the initiative in meeting training requirements, and must be prepared to bear the risks and frustrations of the changeover period.

Once this changeover period is nearing an end, management may have to face the problem of job satisfaction and wages. Will an operator who has the ability and has assumed new responsibilities be satisfied with his original wages, or will he want the wages and benefits of the more skilled laborerers? Will the duties of a skilled technician be challenging enough to keep him satisfied? He will be the plant expert, the man that the utility would be least likely to transfer or want to lose. Will he fit into a regular job so that he can be immediately available as a troubleshooter if full-time maintenance is not merited? The organization that finds itself faced with these problems after the installation of elaborate controls either failed to recognize the situation into which it was entering or neglected to prepare properly for it.

All of this does not mean that a program of automatic and remotely controlled installations should not be established. It simply means that the prerequisites must be realized and achieved. These are, simply, that management must understand the probem, be enthusiastic about and have a real desire for controls, and be willing to provide the means and personnel required to keep the equipment operating.

Wise Planning

A water utility can install automatic and supervisory controls wisely and with full knowledge of the situation that will arise through their use. In the beginning, the primary justification for the equipment was more than likely, better operating performance of the system, rather than a reduction in operating cost. A certain fallacy, that automatic and supervisory control is primarily installed to reduce operating cost, seems to exist. This may or may not be true. It depends upon the number of stations involved, the character of the stations, and to a very large degree the competence of maintenance personnel. Wise employment of control means selection of control features that are appropriate to the conditions in which they are to be used, for control is not just good or bad; it is good under certain conditions.

This organization's original, attended stations each required three shifts of operating personnel. The new unattended stations require only the attendance of maintenance personnel. Usually such maintenance may be accomplished during a single shift, and this maintenance shift may attend several stations.

The attended stations required considerable expense and facilities for operator comfort such as additional heating, in some cases air conditioning, washrooms, locker rooms, office space, and office equipment. The unattended station may be constructed without any of these, except for ventilation and the small amount of heating that may be required to prevent freezing.

The original stations each were individual control centers from which only a part of the overall system could be analyzed and controlled. Under those circumstances management's information on and control of the complete system were difficult to maintain because all data and directives had to be channeled back and forth by some form of voice communication. The unattended stations, supervised and controlled from a central control center, give management immediate information on all important system conditions, and provide virtual fingertip control of every component of the system. Such information and control obviously contribute to more efficient operation by permitting an up-to-the-minute analysis of the complete system. The analysis may point out certain trends, allowing management to anticipate a condition in a particular area. The most capable and efficient means of meeting that condition may then be judged. Such judgment would be important in establishing additional power demand charges at a particular station. It might be that reserve pump capacity at another station could more economically serve. The immediate knowledge of system conditions provided by suitable remote instrumentation permits an earlier dispatch of general maintenance crews to troubled areas, as in station failures and large main breaks.

Proper Approach

It is reasonable to expect that there will be more and more automatic equipment and unattended stations. The trend is well established and there will be no turning back. It seems wise to enter this era of more complexity with caution, and not to get into such a position that there is complete reliance upon equipment without preparedness for its service and maintenance.

The operation of a complete water distribution system from a single, centrally located control center obviously is attractive to both large and small organizations, perhaps even more so in smaller communities, although their preparedness and maintenance facilities are generally poorer. Even so, engineers will be called upon by both large and small communities to prepare plans for unattended stations. Certainly it is the engineer's duty to inform management of the consequences of the use of such equipment. Also, he should persuade management to prepare adequately for maintenance, and provide training of operators. Management can do a great deal in promoting a receptive attitude in operating personnel by keeping them informed and permitting a limited degree of participation in control system planning.

The engineer must try to recognize the qualifications of the organization

and then select the most appropriate control scheme. The range of selection extends from near-manual to completely automatic systems. Somewhere within those boundaries is the most practical solution for any particular organization. More safeguards must be provided for the less skilled operating staff, and the cost, therefore, will be proportionately greater.

Consider, for example, a booster pumping station, beyond which lies a dispersed well field, all to be controlled and supervised from some central control center. A very reliable control scheme would be a separate and completely independent control system for each well pump and booster pump, together with individual interconnections for remote instruments. Immediate maintenance would seldom, if ever, be required to operate continually from the central control station. Such a control system obviously would be quite expensive even if the distances involved were only a few miles.

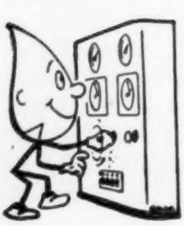
The system could be made still more reliable by underground, direct-wire circuits at even greater cost. A considerably less expensive system that might be used would employ only a single paired telephone line connected in common with all well and booster pumps. This same two-wire interconnection would permit continuous remote instrumentation, together with remote control of all well and booster pumps. Then, however, immediate and first-class maintenance would be necessary even to approach continuity of operation from the central control station.

More reliability, at additional line cost, could be obtained by separating control and instrumentation, thus providing the operator, through instrumentation, with information on system conditions, although he may have lost control. Similarly, he could have control even when instrumentation fails. Still greater reliability, again at more cost, would be realized by separating the well field and booster pump controls. The formation of such blocks of control, each separate and independent, is a means of safeguarding against complete loss of control. The cost of equipment and signal channels is proportional to the number of control blocks, although the necessity for these blocks will be some inverse function of the caliber of maintenance available. In this respect, there are advantages in procuring equipment which can all operate over a single line, and installing it initially to operate over several lines to form individual blocks of control. Then, as maintenance and care of the equipment becomes well established, signal channels may be abandoned by grouping more of the control on a single line.

Other schemes for lessening the disadvantages and seriousness of control failure are many; none is perhaps so important, however, as the incorporation of failure-safe features. In other words, the control system must not only be designed to function properly under normal conditions, but it must also work properly during abnormal conditions. For example, the designer must always consider what would happen in the event of power failure and what would happen when power was restored. Failures of control components, such as malfunction of limit switches, burned-out coils or contacts, and sticking solenoids are but a few possibilities. The performance of the equipment under many of these mal-

functions can be predicted and influenced by careful design. The extent to which a failure-safe control system should be employed depends upon the seriousness of the possible results of failure. There, the control designer's problem becomes a problem of judgment, because the simplest control system is the best and a control system that is completely failure-safe cannot be simple. An absolutely foolproof control system is akin to perfection and, therefore, will seldom, if ever, be realized.

It must be admitted that the rewards of automatic and supervisory control are many; its characteristics, however, must be understood and its employment must be recognized as much more than the utilization of just another piece of machinery. The important question about installing controls is not whether it can be done but whether this is the place to do it. The use of automatic and supervisory control equipment may require a reshaping of water-utility operation if its benefits are to be fully realized.



Progress of Controls and Instrumentation at St. Louis

Frank E. Dolson and Hadley A. Quade

A paper presented on Apr. 18, 1958, at the Conference on Control, Instrumentation, and Automation in the Water Supply Field (cosponsored by the Missouri Section, AWWA), at the Univ. of Missouri, Columbia, Mo., by Frank E. Dolson, Vice-Pres. of Distribution, and Hadley A. Quade, Staff Engr., both of the St. Louis County Water Co., St. Louis, Mo.

CONTROLS and instrumentation leading to a high degree of automation are attaining a position in water distribution systems second only to that of hydraulics. Ways are being devised to make better use of the underground piping system, which often represents as much as 50–70 per cent of the total amount invested in public water supply facilities. Some engineers are designing fully automatic booster pumping stations to increase the capacities of arterial mains for short periods of peak water demand. Other designers, by placing finished water storage in surface tanks at strategic points and by depressing system pressure gradients below normal during the fill cycle, are achieving unusually good 24-hr load factors on feeder mains. Often surface storage and pumping combined with booster pumping on the supply main will increase capacity by more than 50 per cent at much less cost than any other method and provide satisfactory reliability.

Through the use of properly designed controls, stations can be made fully automatic. They may also be remotely controlled from a central load dispatching center during critical periods and operated on a self-contained, fully automatic basis at other times. Proper combinations of pressure and flow-sensing devices, sequencing equipment, interlocks, time clocks, time-delay mechanisms, multiple-purpose valves, low- and high-pressure stats, leased wires, failure-safe circuitry, and other devices now available provide the designer with many possible ways in which functional requirements can be met. Valve closures, timed or proportionate to an increase in pressure or water level, can reduce or eliminate water hammer hazard.

The pattern of water use in fringe developments is different from that in older cities which have a concentrated land occupancy and industry in proportion to population. Ratios of 30 to 1 or more between the maximum peak-hour rate on hot summer days and the minimum night rate on off-peak days may often be observed. But more important to the distribution system designer is the variation between the maximum and minimum rate of usage on a maximum day. Often this ratio may be 10 to 1 or more. Little can be done to reduce this ratio in strictly residential areas. Some relief may be obtained by effective advertising or public relations efforts that stress the

futility of peak-hour sprinkling and the desirability of nighttime soaking of lawns and shrubs. The load factor on pipe supplying local distribution cannot be changed materially, but much can be done to improve the use ratio of the arterial and primary mains system. Although a unity load factor on supply mains is difficult to achieve, it may be approached by diversely locating balancing storage near points at which it is needed. The quantity of storage at any one point should be closely related to the needs of the area which it serves.

If maximum benefit is to be obtained from such storage, it must be used effectively. Uncontrolled use of water from elevated or surface tanks during off-peak periods may result in insufficient reserve to handle peak-hour requirements. If the supply main in this situation is only large enough to handle about 120 per cent of the average daily flow on design days, service outage or reduced pressure may be expected. It is equally important to control the rate at which depleted storage is refilled. Where storage is scattered throughout the entire system in small units, predetermined back-pressure control must be exercised if storage units more remote are to be completely and not just partly refilled. Only by apportioning the available capacity of the transmission mains to the different storage facilities can optimum and most economical use of the system be obtained.

St. Louis County System

It is a purpose of this paper to discuss the progress that the St. Louis County Water Co. be made toward fuller or better load factor use of its primary main system by the employment of special equipment and controls. Two installations which are illlustrative of all others in the system will be described in detail. These units, which are fully automatic within themselves, serve only a particular portion of the system. One of them, at certain times, is correlated with one of the three main plants, although for the most part its operation is controlled by pressure or water level at other strategic points.

The second unit which will be described is locally controlled and its operation is not related to any other unit except by manual resetting of the control functions. Automation, in its true sense, has not been accomplished, in that there is no provision for automatic resetting of controls.

A brief description of the distribution system and a few of its salient features will aid in the understanding of why controls and equipment are so important in its operation.

Through a network of 1,750 miles of pipe, the system supplies water service to 140,000 customers residing in a 210-sq mi area that borders the city of St. Louis on the north, south, and west. Three municipalties purchase water for resale through their own distribution systems to 17,000 additional customers. The city of Kirkwood, with a population of about 25,000, purchases water for part of its peak requirements from the company under a special rate that takes into consideration the unfavorable load factor of this type of service. The county system serves directly or indirectly more than 157,000 customers or slightly more than 600,000 people.

These customers use almost 48 mgd on the average. Under extremely high temperatures and prolonged drought conditions, maximum-day water requirements may be approximately 115 mgd and peak-hour usage may be at the rate of 195 mil gal.

Supply to the system is provided by three plants. The largest of these is located on the Missouri River and has a capacity of 85 mgd. The second largest, located approximately 15 mi downstream on the Missouri River from the first, has a capacity of 36 mgd. A third plant, which is the newest and smallest of the three, has a capacity of 15 mgd. The combined plants have a total capacity of 136 mgd.

The terrain of the service area has ridges which gently slope into numerous valleys and creeks, separating the watersheds of the three rivers that form the boundaries for most of St. Louis County. The difference in elevation between the bottom land adjacent to the rivers and high points along the ridges varies between 250 and 350 ft. This variation in relief poses many problems in maintaining proper pressure to concentrated groups of customers residing at the extremities of these elevations.

Except for two large areas, which contain only a small proportion of the total number of customers, service is rendered through what is normally considered a single pressure system. One of the separate pressure districts, known as the Ellisville district, has a maximum requirement of 1 mgd. The other, known as the Sunset-Mehlville district, has a maximum requirement of 5–6 mgd.

There is now installed within the system 26 mil gal of finished-water storage. Of this total, 10 mil gal is in a centrally located reservoir that receives the entire output of one system of transmission mains. All water is repumped at this reservoir, and pressure control in the single pressure system is adjusted and maintained by staffing the pumping station on a 24-hr basis. Because this station is subject to manual control, data resulting from its operation are excluded from this presentation.

The remainder of the storage is scattered throughout the system at points of high elevation in four 250,000-gal elevated tanks, three standpipes having a total capacity of 2.3 mil gal, and six steel surface tanks, having a total capacity of 12.5 mil gal. In addition, Webster Groves, one of the wholesale customers, has storage of 1.9 mil gal.

Each surface tank is equipped with pumps having sufficient capacity to deplete its storage between the hours of 5:00 and 10:00 PM on a maximum day. Most of these pumping stations have the capacity to pump at the rate of 14 mgd.

The size of the service area and the magnitude of the topographical relief require extensive use of booster and tank pumps to meet service requirements on hot summer days at minimum cost. Excluding pumps associated with storage tanks, as many as fourteen booster pumps at eleven different locations are often used simultaneously in the distribution system on such days.

The amount of water used by industry in this system is small compared to residential usage. The variation in the rate of water use at different hours during the day and night is typically residential. In the summertime the rate of usage peaks between 7 and 9 PM and it recedes rapidly thereafter to only 30 per cent of average during the early-morning hours. On design days the ratio between maximum and average rate of usage is 1.75 and the ratio between maximum and minimum rates of customer demand reaches 6.0.

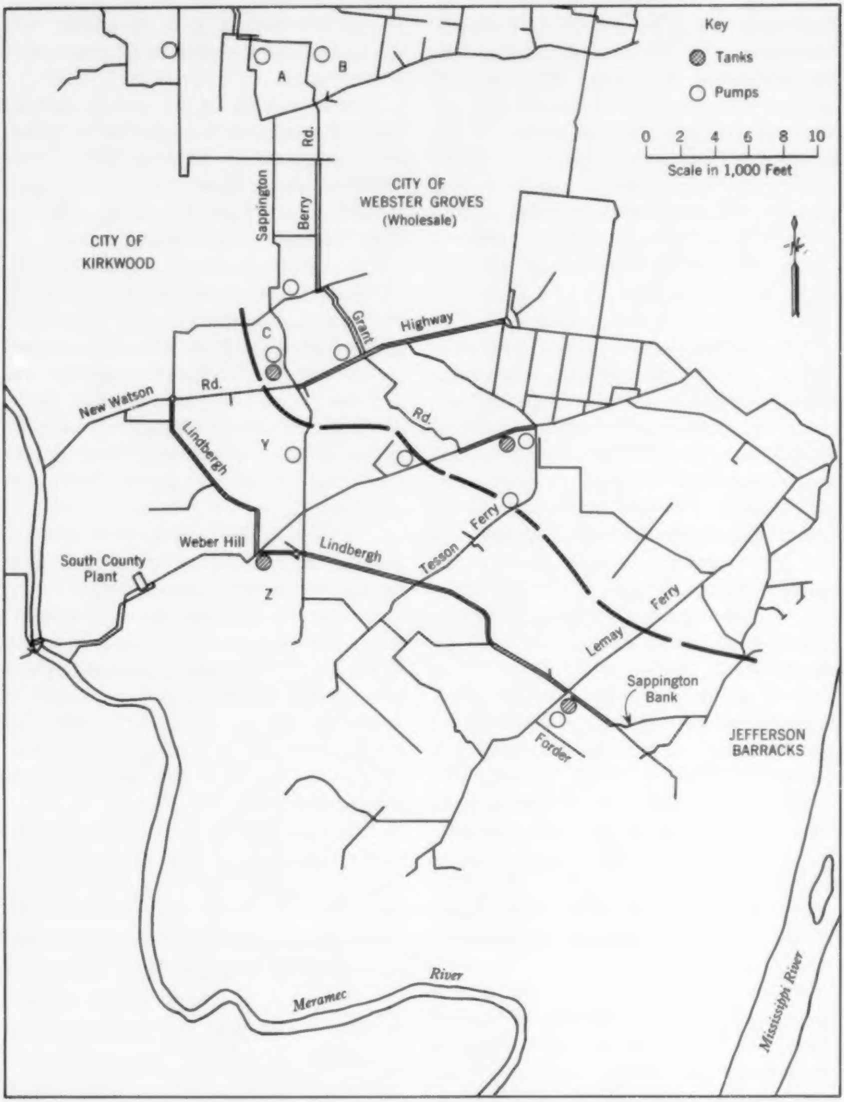


Fig. 1. Locations of Pumps and Storage Tanks in St. Louis County

Points A, B, C, Y, and Z represent the Bennett Pump, Des Peres Pump, Sappington Tank, Sappington Road Pumps, and Sunset Tank, respectively.

Two other factors which merit mention because of their influence on design are rapid growth and unstable zoning. Of these, problems resulting from changes in zoning are by far the more serious. Lack of zoning, piecemeal or fragmented planning, or changes in previously established land use patterns are serious impediments to economical water system design. Unanticipated incremental increases in water requirements often make necessary the installation of feeder mains through developed sections at greatly increased expense. Often water mains of adequate size to supply original requirements must be paralleled with another pipe to provide for the increased needs caused by changes in zoning. In such cases, more water tanks and pumping stations may be needed too. If the ultimate land use pattern had been known, proper capacities could have been incorporated in the original design at much less cost.

Design Criteria

In the establishment of design criteria, physical features of the system, loading characteristics, rate of growth, and zoning, or the lack of it, all have some influence. The basic criterion of design is the provision of the most reliable service possible in the most economical way. Gadgetry and the use of unnecessarily elaborate controls or circuitry have been discouraged. Innovations, if used, must measure up by being better and more economical than other methods; otherwise they are not incorporated into the final design.

Where reliability is not involved and where the economics of the situation cannot be determined, a long-term design policy must often be established. In newly developing areas, where the pattern of land use may be subject to change, it is often prudent to install a small-diameter supply main initially. As the pattern of land use becomes known, and when the need for additional service materializes, a reinforcing main of proper size may be laid over an alternate route. Decisions like this relate more closely to policy than to design standards.

Some of the policies and criteria of design relating to distribution system expansion and operation in this system are:

1. The combined capacity of the treatment plants and associated pumping facilities is based upon maximum possible day requirements with sufficient margin of excess capacity to allow time for the design and construction of additions in advance of actual need. On design days, the load factor on these plants and the transmission mains to the first point of connection to the distribution system should be close to unity.

2. The capacity of transmission pipelines lying between the pumping station and the first point of connection with the distribution system is based on average flow over a 24-hr period on maximum design days.

3. Clear-water balancing storage in sufficient quantity to provide for variations in hourly demands on design days should be located throughout the distribution system near points at which it is needed. By providing storage within the distribution system near the point of ultimate use, instead of clear-water storage at the main pumping stations, reliability can be increased and important savings can be effected

in capital investment and operating expense.

4. Except in cases where the elevated storage, riding on the line, is necessary to meet local conditions, surface tanks for balancing storage should be considered. With this type of storage, system pressure gradients, by the use of properly designed equipment and controls, can be reduced below normal during the refill cycle and thus insure maximum utilization of supply mains at night, when customer demand is at a minimum.

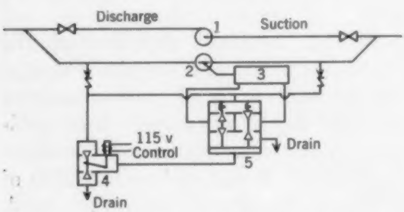


Fig. 2. Hydraulic Schematic Diagram of the Bennett and Des Peres Booster Pumps

This design is essentially the same for both pumps. The components are: 1, submersible pump; 2, butterfly valve; 3, operating cylinder; 4, three-way solenoid valve; and 5, four-way valve.

5. Booster pumping is economical and may be used advantageously for transferring water from one pressure district to another, furnishing peak hour requirements by increasing flow rates in strategic mains, and increasing night rates in supply mains to fill balancing storage.

6. Multiple supply pipes installed over alternate routes are preferable to a single supply pipe when the ultimate pattern of land development is unknown or the rate of growth makes it economical to delay final improvements to a later date.

7. Failure-safe features should be incorporated in all control designs.

8. Valve closures which interrupt high flow velocities should be slow and subject to adjustment. Proportionate band closing on altitude valves installed in the supply pipes to storage tanks is preferable to any other method of control.

The discussion that follows will deal with controls and instrumentation of two specific installations. Both are located in the southern part of the distribution system.

Points A, B, and C in Fig. 1 show the locations of two submersible booster pumps and a large surface tank with an associated pumping station, all of which are interrelated in operation. The controls that cause these facilities to be complementary to each other are the first example considered. Also shown in Fig. 1 at Points Y and Z are two other facilities which will be the second example. In operation, these two are dependent upon each other.

At Points A and B in Fig. 1, large booster pumps have been installed in each 20-in. pipeline. Each pump, a submersible turbine unit driven by a 200-hp electric motor, is mounted in a vertical steel barrel which is installed in bypass piping around a 16-in., hydraulically actuated, but electrically piloted, butterfly valve in the 20-in. supply line. The motor pump unit operates submerged in the flooded steel barrel. Entry of water into the motor, which runs in oil in a watertight housing, is prevented by an ingenious mercury seal on the motor shaft. The pumping unit has a rated capacity of 6,000 gpm at a 105-ft head. The head capacity is such that the pump may be operated through a wide range at

little sacrifice in efficiency and with no danger of overloading the motor.

Without pumping, the two 20-in. supply pipes have a combined capacity of 10–12 mgd on a load day. This is increased to almost 19 mgd when the pumps are in operation.

The facilities at Point C, Fig. 1, are known as the Sappington Road Tank and Pumps. There is a 2.5-mil gal, all welded-steel surface tank, 92 ft in diameter and 50 ft high. Fill and draw to and from the tank is through a single 20-in. pipeline on which a 10-in. multiple-purpose butterfly control valve has been installed.

Four identical, close-coupled pumping units, each having a capacity of 2,500 gpm at 91-ft head, take suction from the tank and pump around the control valve into the system. With all pumps running, water may be withdrawn from the storage tank at the rate of 10,000 gpm, or 14.4 mgd.

An antivortex device, constructed over the entry of the supply pipe into the bottom of the tank, allows the withdrawal of all but the bottom 6 in. of water.

Submersible Pumps

The purpose of the submersible pumps is to maintain adequate supply to the Sappington Tank area during off-peak day hours, and to replenish storage during the night.

The control functions required for this installation are: [1] to maintain a pressure above minimum requirements at the Sappington Tank location during the day from 6:30 AM to 5 PM; [2] to refill the tank during the period from 10 PM to 6:30 AM, with tank water-level control of the pumps to avoid water hammer; [3] to be off during peak hours from 5 to 10 PM when the Sappington Pumps supply the load; [4] remote control of the submersible pumps from the Sappington Tank; [5] time-delay starting of the second submersible pump; and [6] high-pressure override located at the submersible pump sites in case other controls malfunction.

A schematic diagram of the hydraulic control is shown in Fig. 2. The submersible pump is installed in a bypass around the butterfly valve. As shown, the pump is not running, the butterfly valve is open, and the solenoid pilot valve is deenergized. When the control calls for the pump to run, an interlock on the pump motor starter closes the circuit to the solenoid pilot valve. With the solenoid energized, sensing water pressure is on the diaphragm of the four-way valve, and the four-way valve shifts. Water then flows to the opposite end of the cylinder operator, and the butterfly valve closes.

Figure 3 is a schematic diagram of the electrical control for the submersible pumps. The control circuit to the right is at the Sappington Tank, where normal control of the submersible pumps occurs. The control signal passes over a leased signal channel to the control circuit at the Bennett Avenue Pump on the left. The control circuit at the Des Peres Pump is a duplicate of the Bennett control circuit. During the day, from 6:30 AM to 5 PM, time switch T-2 is closed, and pressure switches P-2 and P-4 control the pump operation. If the system load is such that P-2 and P-4 close the circuit, the Bennett Pump starts; then, after the time-delay relay (TD) is satisfied, the Des Peres Pump starts. The pumps will stop if either the pressure or time switch opens. From 5 to 10 PM both time switches are open and the pumps cannot run. From 10

to 6:30 PM, time switch T-1 is closed and pressure switches P-1 and P-3, sensed from the tank pressure or level, control the pumps; and if water level has been lowered, P-1 and P-3 are closed, and the pumps start when time switch T-1 closes. The pumps do not start simultaneously, however, because of time-delay relay (TD). When the tank is almost full, P-3, and then P-1, open to stop the submersible pumps. With the pumps off, the system gradient is high enough to fill the tank, although at a reduced rate. Two safety controls are in each of the circuits at the pump location. One is a pressure control, which opens if the pressure exceeds normal operating pressure at the submersible pump site. The other is a relay controlled by an electrode in the submersible motor casing. If the motor should leak oil and if water should rise in the casing, the relay would open when water reached the electrode and the motor would not start, or would shut down if operating.

Figure 4 is a view of the Des Peres Pump site which shows the top surface of the vault that houses the hydraulic equipment and the electrical equipment mounted on a service pole. Figure 5 shows the interior of the control cabinet.

The effectiveness of the installation is illustrated in Fig. 6, which shows pressure charts from the Sappington Tank and one submersible pump location for the same period of time. At 11:40 AM the pressure at the Sappington Tank dropped to 28 psi, and the Bennett Avenue Pump started. The pump ran until 5 PM, when it was stopped by the time control. From 5 to 10 PM the pump was off, and water was supplied at a rate of 7.6 mgd to the system from the Sappington Tank. Because water was used from the Sappington Tank, the pressure control, sensed from the tank water level, was closed and the submersible pump started when the time switch closed at 10 PM. It then ran until 5:30 AM when the water level

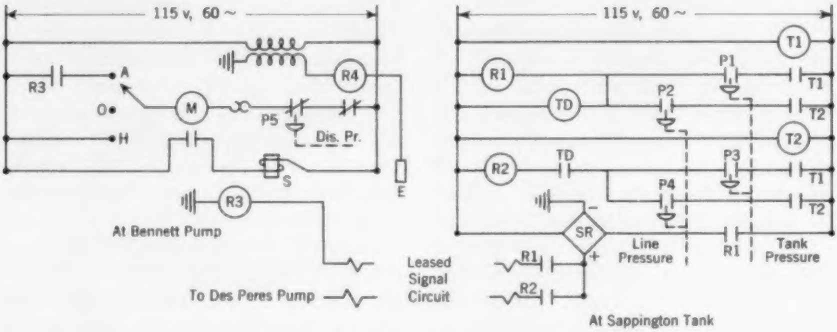


Fig. 3. Control Schematic Diagram for Bennett and Des Peres Pumps

The Bennett control circuit is shown; the Des Peres circuit is identical. Key: T1 and T2—time switches; P1, P2, P3, P4, and P5—pressure switches; R1, R2, R3, and R4 relays; TD—time delay relay; SR—selenium rectifier; M—motor starter coil; S—solenoid valve (butterfly valve control); E—motor electrode.

Fig. 4. Des Peres Pump Site

The hydraulic equipment is housed in the vault, the top of which is visible in the picture. The electrical equipment is mounted on the service pole.

in the tank opened the control circuit.

Sappington Tank

The purpose of the Sappington storage tank and pumps is to supply water to the surrounding area during peak hours and to utilize the off-peak capacity of supply mains for refilling the tank.

The controls can be divided into two categories: pump controls and tank valve controls.

The functions of the pump controls are to maintain adequate pressure in the surrounding area during the load period, but particularly between the peak hours of 5 and 10 PM, and to provide low water-level cutoff.

The functions of the tank valve controls are to cause the butterfly valve to [1] allow the tank to fill during the night period, [2] maintain a set minimum pressure, or above, on the distribution system during fill period, [3] act as an altitude valve by closing when the water elevation reaches a predetermined level (the closing is timed to prevent surges or water hammer in the system), [4] act as a check valve when the pumps are operating, [5] close in case of electric-power failure or sensing-pressure failure (fail-safe function), and [6] close in proportion to the increase in water level during the last 5 ft, or other designated height, of fill.

Originally, the sixth function above was not included in the valve specifications because it was not necessary for correct operation of the installation. When the Sappington Tank was first installed, the entire distribution system was supplied by one plant, now called the Central Plant. There were then three elevated tanks more remote from the plant than the Sappington ground storage tank, and system gradients were such that the Sappington Tank would fill first. This eliminated the possibility of a surge on the system. With the growth of the county, the South County Plant was built. The

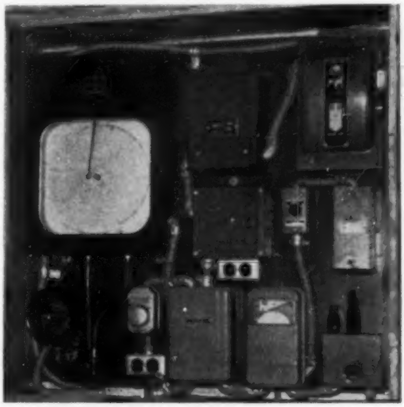


Fig. 5. Control Equipment at the Des Peres Pump Site

This is an interior view of the box mounted on the pole shown in Fig. 4.

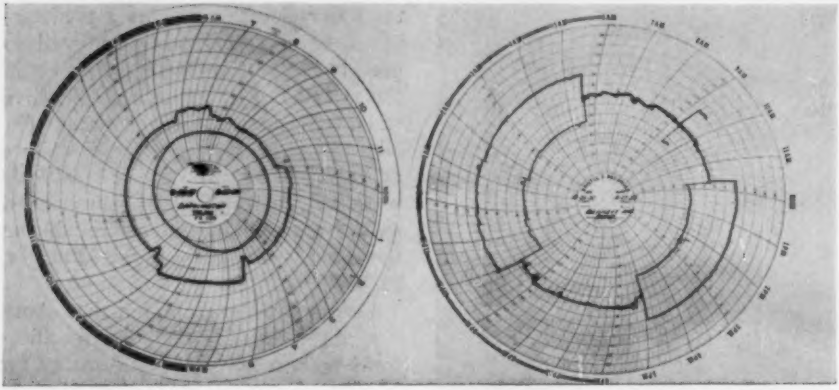


Fig. 6. Pressure Charts for the Sappington Tank and Bennett Pump

These charts demonstrate the effectiveness of the control installation. Left, Sappington Tank; right, Bennett Pump.

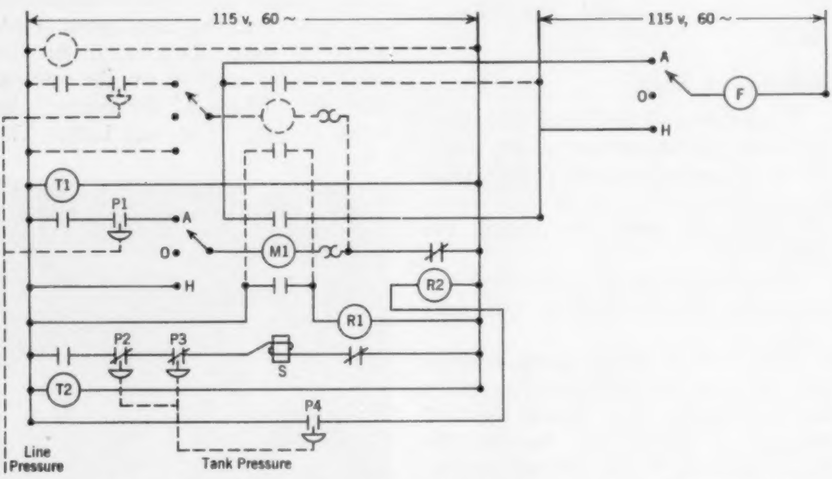


Fig. 7. Schematic Diagram of Controls for Sappington Tank and Booster Station

The controls for one pump are in solid lines. The controls for a second pump are in dashed lines. Other pump controls would parallel those for the second pump. Key: T1 and T2—time switches; P1, P2, P3, and P4—pressure switches; M1—motor starter coil; R1 and R2—interposing relays; S—solenoid valve (butterfly valve control); and F—vent fan.

Sappington Tank is within the area served by the new South County Plant. During the fill period more than 75 per cent of the South County Plant output may be flowing into the tank. Under this condition a surge may occur when the supply valve abruptly closes. For this reason, closing of the valve in proportion to the increase in water level during the last designated number of feet was added as a control function, and made part of the installation when the South County Plant was built.

The schematic control diagram is shown in Fig. 7. The time control, T-1, and presure control, P-1, are in series, and are connected to the coil of the pump motor starter, M-1, when the selector switch is in "AUTO" position. When the pump operates, an interlock on the motor starter energizes relay R-1, which opens the circuit to the solenoid valve controlling the tank supply valve and the supply valve will remain closed while the pump is operating. Another interlock on the motor starter energizes the vent fan circuit to keep the ambient temperature of the booster station at a safe value. Pressure control P-4, sensed from the tank water pressure, is normally open, but it will close if the tank water level gets too low. When P-4 closes, relay R-2 opens the circuit to all the pump motor starters, stopping the pumps. Pressure control P-4 resets when the tank refills. Time switch T-2 and pressure switches P-2, P-3 control the tank supply valve. T-2 is set to be closed for the fill period from 10 PM to 6:30 AM. Pressure switch P-2 opens the valve control circuit when the tank is full and tank supply valve closes. Pressure switch P-3 is a safety in case switch P-2 does not function. The second pressure switch is considered essential, since an overflowing storage tank in a residential area cannot be tolerated.

Figure 8 is a schematic diagram for the butterfly valve control as originally installed, and it fulfills the first five functions given above, but not the sixth. The control is both electric and hydraulic. As shown, the time switch, pressure switch, and relay are closed, and the tank is filling. The three-way solenoid valve is energized and sensing pressure is on the bellows of the controller. If the system pressure is higher than the predetermined set minimum pressure of the controller, the butterfly valve opens to a position where the system pressure equals the set point. Thereafter, the sensing pressure modulates the controller four-way valve to hold the system pressure at the set minimum. When the tank is full, the pressure switch, sensed from the tank side of the butterfly valve, opens, and the solenoid valve is deenergized; the sensing pressure to the controller bellows is stopped, and the bellows is relieved to zero pressure. The four-way valve shifts so that water flowing to the hydraulic operator closes the butterfly valve. When a pump starts, a relay in the solenoid valve circuit opens and the butterfly valve cannot open, thus acting as a check valve. On power failure, the solenoid is deenergized and the butterfly valve closes.

Figure 9 is a schematic diagram of the butterfly valve control, as now installed, with the valve closing in proportion to increase in tank water level during the last 5 ft of fill. The control is electric, hydraulic, and pneumatic. The original hydraulic equipment was modified by changing the

sensing bellows from a 30–70-psi water unit to a 0–20-psi air unit, because the pneumatic control is based on a 3–15-psi air signal. Also two pneumatic controllers and an air relay were added. As shown, the time switch, pressure switch, and relay are closed, and the tank is filling. The three-way solenoid valve and the air supply solenoid are energized. If the water level is below the last 5 ft of tank fill, controller 4100-UR has not started to function, and a 15-psi air signal is to the set minimum pressure. With an increase in system-sensing pressure, the air signal is increased and the butterfly valve moves toward open. A decrease in sensing pressure decreases the air signal and the butterfly valve moves toward close. When the tank is 5 feet from full, the 4100-UR controller starts to function. As the water level rises, the air signal output from 4100-UR controller decreases. The air signal from this controller goes to the diaphragm of the air relay. If the

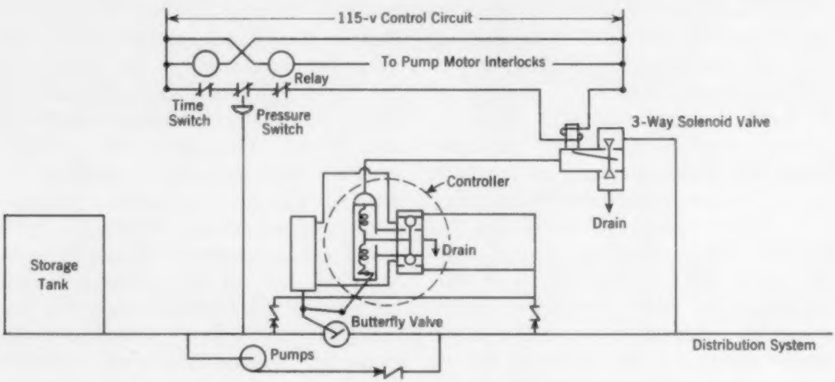


Fig. 8. Original Butterfly Valve Control

This control does not close proportionately to the increase in water level height, and thus permits a surge when the valve abruptly closes. Later installations provided for this possibility.

passed to the upper diaphragm of the air relay. This keeps the valve of the air relay seated, and the air signal from the 4100-U controller is transmitted to the bellows of the modulating controller. If the system pressure is higher than the set minimum pressure, the air signal passes to the bellows of the modulating controller, and the butterfly valve opens to a position where the system-sensing pressure to the controller 4100-U actuates the mechanism to give an air signal equal output signal from controller 4100-UR is greater than the signal from controller 4100-U, the signal to the controller is equal to the 4100-U air signal. If the signal from controller 4100-UR is less than the signal from controller 4100-U, the air relay valve opens to bleed air so that the air signal to the modulating controller is equal to the signal from controller 4100-UR. Since the air signal from controller 4100-UR decreases in proportion to the increase in tank water level during

the last 5 ft of fill, the butterfly valve will close in proportion to the increase in tank water level, if the system-sensing pressure remains above the set minimum pressure.

The three-way solenoid valve is de-energized when the tank is full, if there is a power failure, or if the pump operates. This condition stops the air signal to the modulating controller and vents the bellows to the atmosphere, thus causing the butterfly valve to close.

the butterfly valve was wide open. Consumer usage was high during the first hour of fill because there was negligible flow into the tank. The maximum rate of fill was 7.5 mgd, which is the design figure for the control valve. At approximately 5:40 AM the butterfly valve closed, which caused system pressure to rise to 38.5 psi. Under other load conditions, or if the South County Plant had been in operation, this pressure rise or surge could have been 20 psi or more.

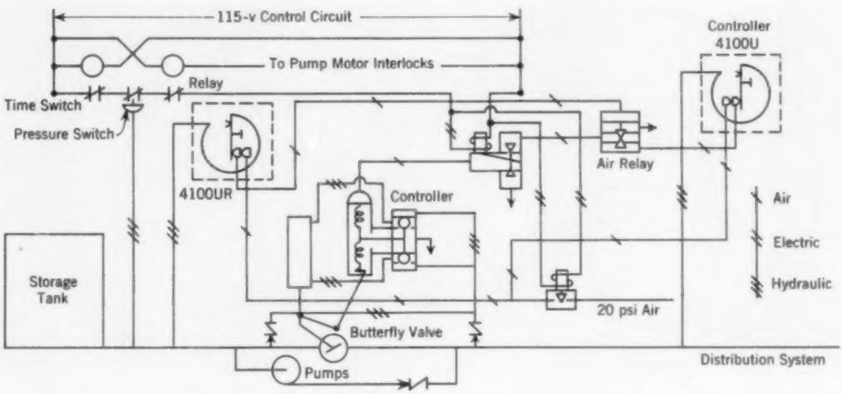


Fig. 9. Present Butterfly Valve Installation

This is an improvement over the controls shown in Fig. 8. Provision for proportionate closing is made.

Operations with and without proportionate closing of the butterfly valve are shown in Fig. 10. The chart on the left is without proportionate closing of the butterfly valve, and the one on the right is with proportionate closing. The chart on the left shows that the tank started filling at 10 PM with the butterfly valve control set to hold a minimum system pressure of 32 psi. The system pressure held steady for 3 hr, then gradually increased 2 psi over the next 4 hr. This was because

The chart for proportionate closing of the valve shows that the tank started filling at 9 PM with the butterfly valve control set to hold a minimum system pressure of 29 psi. There is some fluctuation in the system pressure; it is, however, fairly consistent throughout the fill period. At 4:15 AM the proportionate valve-closing control began to function. The gradual rise in system pressure eliminated the possibility of a surge and allowed time for the plant operator to reduce the rate

of pumpage, thereby preventing excessive system pressure.

Sunset Tank and Sappington Pumps

The Sunset Tank is located at Point Z in Fig. 1. This is a 250,000-gal elevated tank, 40 ft in diamter and 30 ft high, with the bottom of the tank 65 ft above the foundation. Fill and draw to and from the tank takes place through a 12-in. pipeline that is equipped with an 8-in., automatic cone control valve.

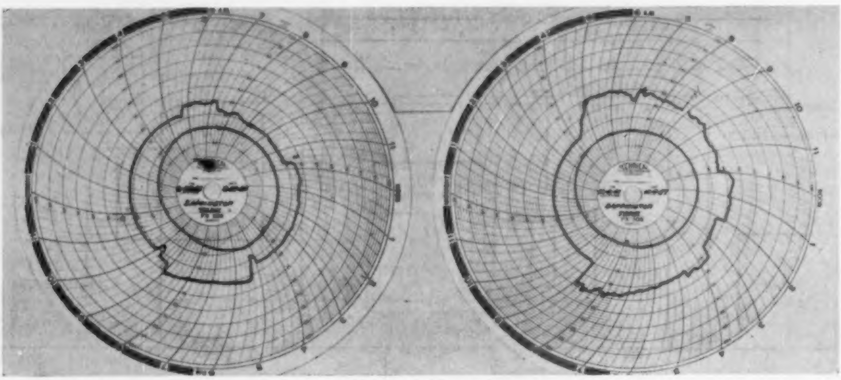


Fig. 10. Comparison of Operation With and Without Proportionate Valve Closing

The sharp rise in system pressure shown on the graph made without proportionate closing (left) could, under some load conditions, represent a serious surge. This has been eliminated in the system with proportionate closing (right).

The facilities at Point Y in Fig. 1 are known as the Sappington Road Pumps. Three identical close-coupled pumps, each with a capacity of 800 gpm at 60-ft head, are installed in a bypass around a 16-in., hydraulically operated, electrically piloted butterfly valve in the 16-in. supply line, and pump water southward.

The control functions of the Sappington Road Pumps, when the South County plant is not operating, are to [1] maintain pressure within preset limits at the Sunset Tank location, [2] sequence depending on the load, that is, with the tank water level, and [3] shut off pumps if controls malfunction (high-pressure override). The control functions, with the South County Plant operating, are to: [1] stop the pumps when the plant starts pumping, [2] open the butterfly valve in the 16-in. line to allow water to flow northward, and [3] start the pumps and close the butterfly valve if the South County Plant cannot maintain pressure at the Sunset Tank.

The schematic control for this installation is shown in Fig. 11. The control circuit at the South County Plant is at the extreme right. To the left above is the control circuit at the Sunset Tank. The remainder of the control is at the Sappington Road booster. There is one leased signal channel from the South County Plant to the Sunset Tank, and two leased channels from

the Sunset Tank to the Sappington Road booster to carry the control signals.

When the South County Plant is not operating, the switch at the plant is open, relays R-7 and R-4 are de-energized, and the Sappington Road Pumps are controlled by the water level in the Sunset Tank. When the water level in the tank drops, pressure switch P-2 closes. The signal is transmitted over one wire of a leased channel, energizing relay R-2, and two pumps start. If the two pumps cannot maintain the water level in the Sunset Tank because of system load, the level drops, and switch P-1 closes. The circuit is closed over the leased channel, energizing relay R-1, and the third Sappington Road Pump starts. As the system load decreases, the water level rises, pressure switch P-1 opens the control circuit, and one pump stops. Continuing rise of the water level opens switch P-2, and the other pumps stop. If the remote controls do not stop the pumps and the pump discharge pressure continues to rise, pressure switch P-4 at the pump site opens the control circuit to all three pumps. P-4 resets when the line pressure drops to a predetermined value.

When the South County Plant is operating, the operator closes the control switch, and relays R-7 and R-4 are energized. The relays R-7 and R-4 are in series, so if relay R-7 closes (lighting the pilot light) the operator knows relay R-4 has picked up. Relay contacts R-4 close the circuit, energizing interposing relay R-5. The normally closed contacts R-5 open, making the Sappington Road Pumps inoperative. At the same time, the normally

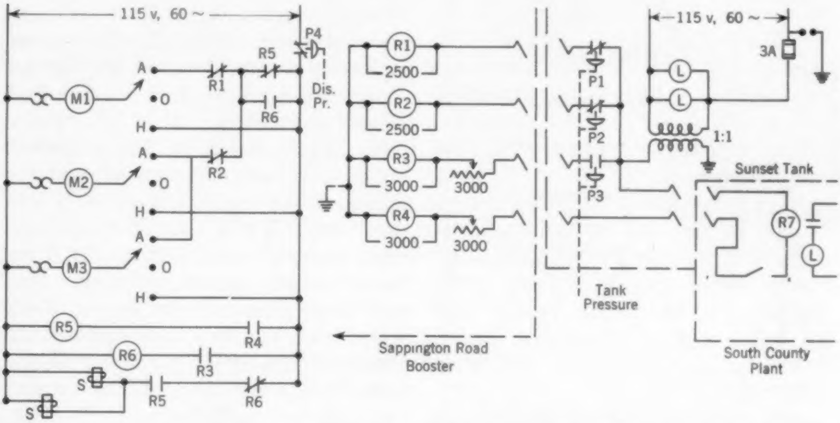


Fig. 11. Schematic Diagram of Sappington Road Booster and Butterfly Valve Controls

The control circuit at the South County Plant is at the extreme right. Directly to the left of this is the control circuit at the Sunset Tank; the remainder of the installation is at the Sappington Road Booster. Key: P1, P2, P3, and P4—pressure switches; R1, R2, R3, R4, R5, R6, and R7—relays; M1, M2, and M3—motor starter coils; S—solenoid valve (butterfly valve control); and L—pilot lamp.

open contacts R-5 close, energizing the solenoid pilot valves which cause the butterfly valve in the 16-in. main to open.

If the system load is very heavy, or if the South County Plant operator

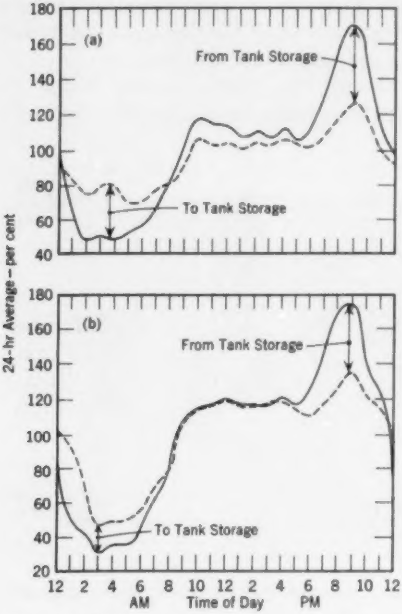


Fig. 12. Effect of Storage on Maximum Day, 1954 and 1956

On both graphs the solid curves represent water supplied to consumers; the dashed curves represent input to the system plant and Stratman Reservoir. The vertical distance between the solid and dashed curves represents flow to and from tank storage—to, when the solid curve lies below the dashed, and from when the reverse is true.

neglects to open the control switch and the water level falls in the Sunset Tank until it is down in the bowl, pressure switch P-3 closes. Relay R-3 closes energizing interposing relay R-6. The normally open contacts R-6 close and the Sappington Road Pumps start. At the same time, the normally closed contacts R-6 open, deenergizing the solenoid pilot valves and the butterfly valve closes. When the Sunset Tank level rises and is almost full, pressure switch P-3 opens. The cycle is repeated until the system returns to normal.

Charts were made of the Sunset Tank and Sappington booster pumps when the South County Plant was not operating. The Sappington Road Pumps were running at 9:30 AM when the charts were changed. The pumps maintained water level in the Sunset Tank until 5 PM, when the system load started increasing. At 7:35 PM the third Sappington Road Pump started. Its capacity was sufficient to meet the increased load and also put water into the Sunset Tank. At 9:55 PM the third pump stopped, and at 10:10 PM the other two pumps stopped. System usage dropped the Sunset Tank water level, and two Sappington Road Pumps started at 11:35 PM to refill the tank. They stopped at 12:30 AM and remained off until 7 AM.

Charts were made with the South County Plant operating. The South County Plant was pumping when the chart was changed at 9:30 AM. Suction and discharge pressures read the same on the Sappington Road chart, since the butterfly valve at the Sappington pump site is open. At 10 PM the South County Plant shut down. When the control switch at the South County Plant was opened, the water level at the Sunset Tank was low enough for the Sappington Road Pumps to run. Also, the Sappington Road butterfly valve closed to cause the differential shown across the pumps. At 11:55 PM the water level in the Sunset Tank was high enough

to shut off the Sappington Road Pumps. The Sunset Tank water level dropped sufficiently to start the pumps at 7:45 AM. The South County Plant started pumping at 8 AM, however, and when the operator closed the control switch the Sappington Road Pumps stopped, and the butterfly valve in the 16-in. main opened.

Charts were made for a day when the system load was heavy and the South County Plant was operating. The South County Plant was pumping when the charts were changed at 9:15 AM. The system load and electric-demand limitation on the South County Plant were such that the water level in the Sunset Tank dropped until pressure reached 30 psi at 2 PM. The low-pressure override switch at Sunset Tank closed, and relays at the Sappington Road Pumps operated to start the pumps and close the butterfly valve. Recovery of water in the Sunset Tank was rapid, and at 3:30 PM the override switch at Sunset Tank opened, returning it to the previous control conditions. At 9 PM the South County Plant stopped pumping, and the operator returned the system to water level control at Sunset Tank. Either three or two Sappington Road Pumps ran from 9 PM to 5 AM, when the Sunset Tank water level was high enough to stop the pumps.

The telephone company has three types of leased telephone channels for telemetering, remote control, or miscellaneous signaling purposes. These are signal, Teletype, and voice. They will reproduce signals in the 0–15 cps, 0–60 cps, and 300–2,700 cps ranges, respectively. The signal channel was considered satisfactory.

For remote control of the equipment at the Sappington Road Pumps, three signal channels were leased from the telephone company. A 60-cycle, 115-v, a-c current may be impressed on the circuit, provided that the signal level is below 60 decibels, and if there is no interference with telephone conversations. The first telephone relays used were too sensitive, and when they were energized they would not drop out, because of the capacity of the telephone circuit. This was overcome by shunting a 2,500-ohm resistance across the relay coil.

The Sunset Tank is at a high elevation and apparently is a good target for lightning. Originally the primary side of the isolation transformer supplying the leased signal channel circuit was fused for 1 amp. The 1-amp fuse blew quite frequently during electrical storms. The 1-amp fuse was replaced with a 3-amp fuse as one step in correcting the trouble. Occasionally the carbon resistor protectors at the terminal of each signal circuit were shorted to ground by lightning. Circuit protectors were installed on the supply, just before the 3-amp fuse and at the service switch. Lightning outages have been materially reduced since these corrections were made.

Figure 10 shows the effect of scattered system storage on two maximum days. In each figure, data on the operation of the 10-mil gal Stratman Reservoir and its associated pumps have been included with the plant output because that station is manually operated.

Figure 12a shows data for a day when customer usage equalled plant, transmission, and arterial main capacities. On this particular day, the customers used 85 mil gal. It should be noted that water from elevated storage started to drain into the system before customer usage reached the 24-hr average. Thereafter, water from

the elevated tanks plus pumpage from the surface tanks supplied most of the requirement above the 24-hr average. During the peak hour approximately 25 per cent of the total customer usage was supplied from system storage tanks.

The difference between customer usage and the input by the plant and Stratman during the off-peak night period is the amount of water, expressed as a percentage of the 24-hr average, which refills the storage tank. In terms of flow in the arterial mains supplying these tanks, the difference represents a load factor of near unity.

Mention should be made of the fact that elevated storage, which is generally at an elevation above the minimum design gradient, is often depleted before the maximum-load period unless isolated from the system by manual or automatic controls. It should also be noted that refilling of such storage reduces the effective capacity of the supply mains.

In contrast, surface storage with pumping is not so handicapped. Through the use of proper controls, it can be used whenever most effective. And by depressing system gradients below the daytime normal during the off-peak night period, by means of the equipment and controls, previously described, the use factor of supply mains can be materially increased.

Figure 12b shows the effect of storage on a maximum day when the system capacity is considerably in excess of customer requirements. On this particular day, the capacity of the system was 121 mil gal and customer usage 82 mil gal. Even under this condition, where customer demand could easily have been met by direct pumping at the main station, the use of storage was effective and economical. By using water from storage during the peak hours, substantial saving in power cost was effected.

Conclusion

Although automation in its true sense will probably never be achieved in the distribution of water, principally because of the low value placed upon the commodity being transported and the comparatively little manual attendance required, progress in that direction is feasible and desirable.

Distribution system components, such as surface storage tanks and booster pumps, may be made fully automatic on an individual basis by properly designed controls, and important economies in transmission main investment and operating expense can be effected through their use.

Rapid, Radioactive Test for Coliform Organisms

Gilbert V. Levin, V. R. Harrison, W. C. Hess, A. H. Heim, and V. L. Stauss

A contribution to the Journal by Gilbert V. Levin, San. Engr., Resources Research, Inc., Washington, D.C.; V. R. Harrison, Research Asst., W. C. Hess, Asst. Dean of Research & Prof. of Biological Chemistry, A. H. Heim, Asst. Prof. of Biological Chemistry, and V. L. Stauss, Research Asst.; all of the School of Medicine, Georgetown Univ., Washington, D.C.

As previously reported (*1, 2*) the use of radioisotopes for the detection of coliform organisms has been directed principally toward the development of a rapid presumptive test. Recent reports (*3, 4*) mentioned the possibility of achieving a direct, confirmed test by the radioactive method. During the past year, efforts have been concentrated to this end. This is a report on the status of the work.

Medium

The logical confirmatory medium was brilliant-green lactose bile broth utilizing the 1-C^{14} lactose used in the presumptive-test experiments. This labeled compound became unavailable, however, making it necessary to investigate the use of some other, preferably cheaper, labeled material. The use of formate was investigated because it has been used in a *Standard Methods* (*5*) coliform confirmatory medium (formate ricinoleate broth) and C^{14} formate is readily available at low cost. Radioactive sodium formate, in various concentrations, was incorporated into specimens of standard brilliant-green lactose bile broth (BGB), which were inoculated with known concentrations of coliform organisms. As a result, a sodium formate concentration of 0.01 per cent was selected for use. Higher concentrations than this were found to be toxic to the organisms, whereas concentrations below this rendered the detection of evolved $C^{14}O_2$ more difficult. Another change in method was the use of 2-oz ointment jars (Fig. 1) for incubation vessels rather than the combination aerating, culturing, and gas-trapping device previously used.

Procedure

The details of the test procedure being used are as follows. The BGB containing 0.01 per cent C^{14} formate is mechanically shaken for several hours prior to use. Shaking has been found to reduce nonmetabolic $C^{14}O_2$ to a reasonably consistent level. The shaken medium is apportioned into paraffin-coated planchets. Membrane filters, on which the bacterial content of the water sample has been concentrated, are placed in the planchets. The planchets are then put in individual ointment jars, the lids are screwed on, and the jars are incubated at 37°C for 3 hr. At that time, a

planchet containing several drops of a saturated barium hydroxide solution is added to each jar, and incubation is continued for an additional hour. During this time, the $C^{14}O_2$ evolved from the culture planchet is carried by diffusion to the barium hydroxide planchet, where it is precipitated as the carbonate.

This method has been found extremely efficient in collecting the $C^{14}O_2$, and it makes the operation very simple. At the end of the collection hour, the barium hydroxide planchet is removed, dried over a heat lamp, and counted in an internal-flow Geiger counter. For purposes of comparison, bacterial cell numbers are determined by colony counts from triplicate nutrient agar plates when pure strains are run, and by the standard presumptive and confirmed coliform tests when unknown samples are run.

In order to consider this method for use as a direct, confirmed test, it must first be established that the medium containing formate compared favorably with the standard confirmatory medium. Accordingly, a lengthy series of tests has been undertaken to compare the two media. A routine has been instituted whereby raw water from the Potomac River and samples from various points in the District of Columbia treatment plants are taken daily. These samples are run by the *Standard Methods* presumptive test for coliform organisms. All portions are subsequently inoculate into BGB confirmatory medium and, simultaneously, into BGB to which 0.01 per cent nonradioactive sodium formate has been added. Both sets of tubes are read in conformance with *Standard Methods*. To date, 1,158 pairs of tubes have been compared. There was

Fig. 1. Ointment Jars With Barium Hydroxide and Culture Planchets

Planchets containing cultures are incubated for 3 hr; barium hydroxide planchets are then added to jars and incubation continues for 1 hr. Evolved $C^{14}O_2$ then diffuses to barium hydroxide planchets and is precipitated as carbonate.

agreement between 1,155 pairs. These data will continue to be collected until the acceptability of the formate medium has been thoroughly investigated.

Proof that the formate medium is satisfactory for use in a standard, confirmed test does not mean that noncoliform organisms will not interfere in a shorter, radioactive test with that medium. Some of the noncoliform groups may live long enough to produce detectable gas in the radioactive test, but not long enough to produce a visible gas bubble in the standard test. This problem is being investigated.

Concurrently with these tests on river and filtration plant water, aliquots of the same samples are being run by the direct, confirmed, radioactive method. As these data are accumulated they will be reviewed and compared for consistency. The data will also be analyzed in an attempt to relate levels of evolved radioactivity with numbers of cells. Table 1 is a summary of quantitative information obtained from pure cultures (chilled to induce lag) of *Esch. coli* in the BGB C^{41} formate medium. The data represent triplicate determinations at each cell population in each test. Approximate cell counts were determined by triplicate plate counts. The activities have been presented for ranges of cell counts and the averages and standard deviations are given. The data show a reasonable degree of correlation between evolved $C^{14}O_2$ and numbers of cells.

TABLE 1

Correlation Between Esch. coli *Population and Evolved Radioactivity*

Run No.	Evolved Radioactivity*—*cpm*								
	1–25 Cells	25–50 Cells	50–100 Cells	100–200 Cells	200–400 Cells	400–800 Cells	800–1,600 Cells	1,600–3,200 Cells	3,200–6,400 Cells
1	†	27	64	94	181	400	792	1,625	4,290
2	7		31		106				
3	11	37	89	155	224	449	906	1,820	6,271
4		37	52	135	391	673	1,373	2,283	6,060
5		24	39	93	164	298	768	1,336	
6	16	24	57	96	224	472	858	1,621	
7	15	28	40	105	245	755	1,676	4,000	6,262
8			14	36	109	254	669		
9		15	37	66	167	321	771	1,871	
10					117	247	520		
11					149	285	480		
12					358	488	985		
13						342	762	2,400	5,450
Avg	12	27	47	98	203	415	880	2,119	5,667
Avg deviation	±4	±7	±21	±37	±92	±163	±340	±841	±839

* Average of three replicates without sterile controls.
† Blanks indicate no data available.

Initial work on the river water and filtration plant samples indicates that several organisms do interfere. When

isolated, they do not produce visible gas in the standard method, but do produce radioactive gas in the isotope method. The seriousness of the interference and the possibilities of preventing it are still unknown, as the work is in an early stage.

As a double check, organisms are being routinely isolated from Potomac River water taken from a sewage polluted zone and run by the radioactive confirmed test to determine the behavior of the normal range and populations of noncoliform groups as compared to the coliforms groups. Pure strains of organisms found in water have been obtained from the American Type Culture Collection and are being tested in a similar manner. This work is also in its initial phase. Of some 20 different organisms tested, only two have demonstrated interference. One is ***Pseudomonas aeruginosa*** and the other, isolated from river water, has not yet been identified.

It is planned to attempt to reduce or closely define the interference from noncoliform organisms and to continue the routine collection of the types of data cited above. When this has been accomplished the data will be submitted for consideration of the method as a standard one.

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Design and Operation of the Holland Filtration Plant

Joint Discussion

A joint discussion presented on Sep. 9, 1958, at the Michigan Section Meeting, Grand Rapids, Mich.

Design Features—Thomas B. Robinson

A paper presented by Thomas B. Robinson, Principal Engr., Black & Veatch, Cons. Engrs., Kansas City, Mo.

THE decision of the Board of Public Works, Holland, Mich., to proceed with a Lake Michigan supply project was made only after exhaustive investigations of underground water supply potential had been completed. These investigations indicated a rapidly receding water table in the area of the existing well field and uncovered no abundant supply in any other area. The quality of the well water was poor, being quite hard, high in iron, and at times objectionable in taste and odor owing to a sulfur content.

The city of Holland lies approximately 5 mi from Lake Michigan, just south of and at the inland end of Lake Macatawa. Originally construction of the new facilities south of Lake Macatawa was contemplated in order that the transmission line might serve a fairly large residential development along the southern lakeshore. This location also would have made a transmission line across the Black River unnecessary. Investigations revealed, however, that because of the high sand dunes along the shores of Lake Michigan south of the entrance to Lake Macatawa it would be extremely difficult to get a transmission line through to the lake, and that there was no suitable lakeshore site for a pumping station. For these reasons, the facilities were constructed north of Lake Macatawa.

The decision to proceed with the \$3,000,000 improvement program was made in 1954, the last contracts were let in 1955, and the plant was in operation in early 1957. Actually water was put through the plant before its completion when the city's principal well failed and use of the new facilities was necessary.

Intake Line

The improvement project included a lake intake drum and intake line, a low-service pumping station, a treatment plant, treated-water storage, a high-service pumping station, a transmission line to the city, and distribution system additions. The city had a 500,000-gal elevated tank and a distribution system which were quite adequate for operation from the well field. Because the

new Lake Michigan supply transmission line entered the city from a different direction, however, major additions to the distribution system were required.

The maximum usage prior to the time of design had been 6.8 mgd, but a large, already developed suburban area constituted a potential 25 per cent demand increase. With allowance for further growth and future industrial expansion, the new facilities were designed for a maximum demand of 14 mgd.

The 42-in. intake line extends approximately 4,500 ft into the lake. At its outer end the intake is 45 ft below low lake level and consists of a steel drum with slotted openings at a minimum depth of 35 ft below the lake surface. The inlet velocity into the drum is 0.25 fps at the design capacity of 40 mgd. The pipeline is of reinforced-concrete with rubber-gasketed steel joints. The inboard end of the line, where the water is 10 ft or less in depth, is covered with a rock blanket to protect it from erosion.

One unusual feature of the intake line is that it is designed to act as a siphon when the flow in the line exceeds 20 mgd in order to reduce the construction cost of the intake line. With this siphon, the inboard end of the intake was installed just below minimum lake level rather than below the hydraulic gradient and has a maximum flow of 40 mgd. This was done to keep the intake line submerged at all times. The intake line remains at this elevation all the way to the low-service pumping station, where its end turns down more than 15 ft below the minimum lake level. The intake line is a gravity line for flows up to 20 mgd, but siphon action is required for larger flow rates. Provision was made at the high point on the line for future installation of vacuum priming equipment.

Another unusual feature is at the point where the intake line enters the low-service pump station. The line terminates in a manhole equipped with a stop plate so that the intake can be shut off from the station without a valve. Repairs to a valve on the inlet line would be next to impossible without this arrangement. Because the design involves siphon action and the manhole is actually the high point on the line, the access hatch to the manhole is gasketed and made airtight, and a tap is provided in the hatch cover for a future vacuum priming connection.

The low-service pumping station was designed in two separate cells, either of which can be taken out of service without disrupting the operation of the other. Each cell was equipped with its own basket screen, which can be raised for maintenance and cleaning by permanently installed winches. Three 5-mgd, vertical turbine, electric-motor–driven pumps were installed initially. In addition, a 10-mgd, gasoline-engine–driven, vertical-turbine pump was installed for emergency standby service. The station provides enough space for five 10-mgd pumping units.

The discharge from each pump was connected to a butterfly valve operated by solenoid and time-delay circuits in such a way that it serves both as a check valve and as a surge control device. The motor-driven pumps are all operated by remote control from the treatment plant. The pump discharge header was looped through the station to feed dual 30-in. lines to the treat-

ment plant. Only one of the lines was constructed in the initial program, but the other line will be added when plant capacity is increased.

A platform is provided in each wet well of the station just above the high water level so that a construction pump can be used to drain the wells. There is a piping connection from the wet well to the plant drain line for discharge from the drainage pump.

The station has a traveling crane for handling pumps, motors, and piping. Also, openings are provided in intervening floor slabs to permit use of the traveling crane in handling the stop plate.

Plant Design

The treatment plant is of quite conventional design in most respects, but it differs from many plants in that transfer pumping and above-ground storage are provided and in some other details which will be described later. The layout of the plant is shown in Fig. 1.

Water enters the plant through a flume, which serves a dual purpose as a metering element and a device for providing rapid mix for the chemicals. A parallel flume, constructed but inoperative at present, will be used when the future 30-in. discharge line from the low-service pump station is installed. At present only alum is added to the water before it enters the flume, but provision was made for the future addition of carbon, lime, and fluorides, if and as they are needed. The city is planning to add carbon-feeding facilities in the near future because of some taste and odor difficulties experienced during the past summer. Provision was made for pre- and postchlorination, and chlorine for prechlorination is introduced immediately after the water leaves the flume.

The chemically treated water flows through a 36-in. line to a distribution well at the far end of the mixing and settling basins, where it is split into two portions. Half of it flows to each of the two basins. The basins are equipped for two passes of flocculation and with a baffle, open at the bottom only, separating the mixing from the settling portion of the basins. The basins are covered and insulated with 2 ft of earth. They are not equipped with sludge removal equipment, but the floors are pitched to drainage flumes and outlets for ease in cleaning. Multiple weirs and flumes are installed to hold overflow rates to a minimum. The flocculator drives are located in a dry well between the two basins. At design flow of 7 mgd in each basin, the total detention time is 4 hr; the basin overflow rate is 11,800 gpd/sq ft; the weir overflow rate is 17,700 gpd/ft of length.

Four rapid sand filters of 3.5-mgd capacity each were laid out with a center pipe gallery and two filters on either side in such a way that the filter bed may be expanded almost indefinitely. The filters were designed for a filtering rate of 3 gpm/sq ft, but hydraulically are capable of operating at a 50-per cent overload. The entire filter operation is pneumatically controlled, the actual controls operating at a 3–15-psi pressure; 80 psi is used for valve operation. Palmer sweeps were used for surface wash. Filter wash water is provided from a 1.5-mil gal steel storage tank, which also serves as the plant's filtered-water reservoir.

Transfer pumps of the vertical-turbine type take suction from a header which connects the two small clear wells, one under each bank of filters. The initial installation consisted of three 5-mgd units, but space was provided for a total of four 10-mgd pumps. The transfer pumps operate automatically from the level in the clear well. As the clear well level rises, additional pumps come on at predetermined elevations.

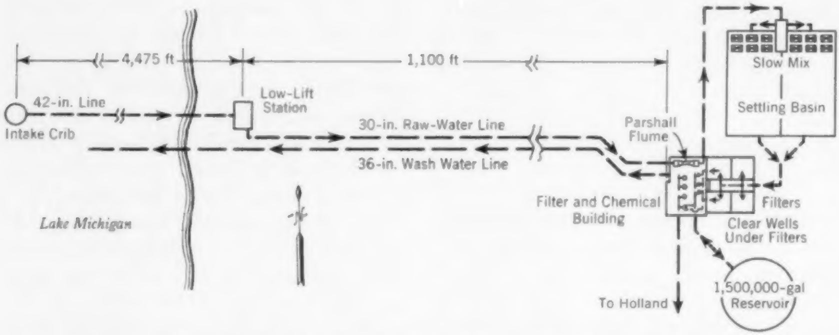

Fig. 1. Schematic Plan and Flow Diagram of Holland Treatment Plant

The water intake and filtration plant are located approximately 6 mi northwest of downtown Holland. The plant is maintained by only seven employees, most of the operations being controlled from the control panel on the operating floor of the filter and chemical building.

The transfer pumps discharge into a header connected to the 1.5-mil gal steel storage tank. This header serves a triple function as transfer pump discharge header, high-service pump suction header, and wash water header.

Transfer pumping is used primarily because of the need to keep the clear well above the ground water table, both to meet state board of health requirements and also to reduce the cost of dewatering. As designed, the settling basins were placed at as high an elevation as possible without putting them on fill, and the very limited clear wells under the filters were just above the water table level when the lake was at maximum elevation. There are additional advantages to transfer pumping, however. The high-service pumps have positive suction head at ground level and do not need to be placed in a pit, where they would be subject to flooding. No separate wash water tank or wash water pump is required, as the steel filtered-water reservoir provides plenty of water at proper head for washing the filters.

The high-service pumps, which take suction from the three-function header, discharge into a header connected to the 36-in. transmission line to Holland. The water leaving the plant is metered by means of a venturi meter located on the plant ground. Three horizontal, electric-motor–driven centrifugal pumps of 3-, 5-, and 7-mgd capacity, and one gasoline-engine–driven pump of 10-mgd capacity were installed. The engine-driven unit is

designed to pump either from the steel tank or directly from the clear well, and may therefore serve as a standby for both the transfer and high-service pumps. The station is laid out to accommodate four 10-mgd electric-motor–driven units in addition to the present engine-driven pump.

As is usual with long transmission lines, it was difficult to obtain pumps with characteristics suitable for operation at low capacity and friction head that would also be suitable for operation at higher capacities, in parallel with other units and at much higher friction heads. The problem could have been solved by installing a variable-speed unit, but cost analyses indicated it would be more economical to provide an extra rotating element, designed to operate at higher head conditions, for each of the two smaller high-service pumps. The lower head units can be used in the winter under low-demand conditions and the higher head units can be installed, if desired, for summer, high-demand use if it is necessary to use these smaller pumps in parallel with the larger pumps. When the extra rotating elements are installed in these units, the firm capacity of the high-service pumping station will be increased.

The pump room that houses the transfer and high-service pumps is at ground level. All piping serving these pumps is at basement level below the operating floor. An overhead traveling crane is provided to serve the entire pump room area, and access to the piping below the floor can be obtained through removal of the precast, concrete slab flooring.

As the plant is served by a single source of electric power over a single long transmission line, it was deemed necessary to provide gasoline-engine–driven pumping units. Certain other functions of the plant require electricity, however, and therefore an auxiliary generating unit was provided. A separate 120-v, d-c lighting system will turn on automatically in case of power failure. This permits the operator to get to the emergency generator unit from any place in the building safely and rapidly. After starting the auxiliary unit, the operator can return to the control panel and manually switch the emergency a-c distribution panel from the normal a-c supply to the emergency supply from the auxiliary generator. All essential features, such as heating, chemical feeders, slow-mix drive equipment, and compressed-air supply, and approximately 50 per cent of the lighting fixtures and outlets, including those in the laboratory, can be operated from the emergency a-c distribution panel.

When a new generating plant being built a few miles north of the Holland water plant is completed, the plant may be able to get an additional source of power, and the emergency standby features will not be so vital.

The transmission line to the city, approximately 6 mi in length, was constructed of 36-in. prestressed concrete. The line includes relief valves and blow-off valves as required and has no unusual design features with the exception of an aerial crossing of the Black River. The aerial crossing was decided upon when it was found that an unstable muck extended over 40 ft beneath the river bottom. The crossing consists of two 110-ft spans of 36-in. steel pipe. These are supported on a central pier and two abutments built on 60-ft piling.

Operation—*James Hornung*

A paper presented by James Hornung, Supt., Water Supply, Water Treatment Plant, Holland, Mich.

It is natural to expect many difficulties in a new filtration plant, and to worry about whether the designers have left all future operators with some unsolvable problem. Those at Holland can honestly say that the initial operation of its new plant was about as free of problems as possible. In fact, the only real changes that were made in the original plans were additions that the designers considered unnecessary expenses but which were made to increase the plant's efficiency. These will be referred to later. A good job by the resident engineer contributed greatly to the project's smooth operation, for he had what is an important qualification for a good resident engineer—to be able to read the designers' specifications. An interesting sidelight above the designers is that they have returned intermittently to see how the operation is proceeding and ask for suggestions for improvement. To most operating personnel, this has seemed a sound approach to practical design problems.

Personnel Training

Because Holland had always had only a well supply there was no one with the proper qualifications available locally to operate the new filtration plant. The superintendent of utilities obtained from the Michigan Department of Health a list of men qualified to operate the plant and sent a job description to each of those on the list. The author was hired about 3 months before the anticipated date of completion of the plant and was given complete charge of all phases of the operation—from ordering equipment to hiring and training personnel.

When other operating personnel were needed, no experienced help was available in Holland. Therefore, the only qualification that was required was high school graduation and completion of a course in chemistry. Operators were hired a month in advance of the anticipated date of completion of the plant. This turned out to be 2 months, however. During this time, the new personnel studied all the physical aspects of the plant and were given a highly concentrated course in all phases of water treatment. When the plant was finally forced into operation without benefit of filtration, owing to failure of the old city wells, it was on an intermittent basis because the wells could handle the load at night. This proved convenient, however, as no operator had to start out working a shift alone. When the plant finally did go into full-time operation, each man had gained enough experience to give him confidence.

Before being hired the operators were advised that they would be expected to work for a state water license, and all have more than fulfilled this expectation in studying for examinations. It was, and still is, the plant's policy to teach each man as much as possible about his job and about the whole water supply field.

It can be argued, quite justifiably, that such a training program may well produce operators who are qualified for higher-paying positions at other

plants, and that there is a risk of losing them. The benefits of good operation more than balance the possible loss of an operator at some future time. These men have been advised that they will get the utmost cooperation in their efforts to acquire a better license, even if they can improve their position as a result. Well designed equipment is of little value without a qualified operator.

Operator training has consisted of:

1. Daily lectures for a month
2. Written examinations
3. Short courses by the state health department
4. Laboratory experiments
5. Books and literature, which were always available
6. Visits to other water plants
7. Visit to the state health department laboratory
8. Visits to sewage plants
9. Discussions of plant problems and examinations based on them
10. Participation in USPHS short filter run research
11. Attendance at area meetings with other operators.

Operating shifts rotate weekly, and the days off change with each cycle. Operators work alone on the afternoon and midnight shifts. This is where good training pays off. As the plant is somewhat isolated from the city proper, hourly checks are made by telephone or radio with the power plant in town.

Operation Difficulties

Naturally, all of the plant equipment was checked as soon as possible after installation and before it was accepted. A humorous incident occurred when the transmission line to town was tested at 125-psi pressure. A 6-in. inspection plate on a venturi meter, which was buried 4 ft underground, broke loose. This caused a 70-ft geyser that gradually lost pressure until it dumped a few yards of mud and gravel on three trucks that were parked nearby.

One of the things that caused concern before operation began was the lack of a wash water rate meter, particularly because the new operators had never been in a filter plant before. The elevated reservoir serves a secondary purpose as a wash water tank, and although it was not believed that a sand bed and filter bottom could be upset, there was no guarantee that this was true. Practice proved that it would be practically impossible to upset a filter. Wash water rates can be measured simply and accurately by using the drop in reservoir elevation and pumping rates, and a meter was therefore unnecessary.

Another source of concern was that the chemical feed equipment was rated for an ultimate design capacity of 42 mgd, whereas the initial output of the plant was less than a tenth of this rate. In practice, however, the feeders have operated very well at only 10–15 per cent of their capacity.

One addition to the plant, which the designers had considered extravagant, was a sampling system for the laboratory. This was installed at some extra expense, but was well worth the cost. Samples of raw, mixed, settled, filtered, and tap water discharge separately into a laboratory sink. This is just as practical as, and far less expensive than, a chrome and glass visual sampler.

All of the filter control valves are air operated at 80 psi. It was found necessary to install an aftercooler for

the air compressor, as water was collecting in the valve controls. In spite of this collection of water, there was no malfunction of the equipment. The aftercooler was installed without cost to the plant by the manufacturers of the pneumatic equipment. Another difficulty in the air system was the leakage of crankcase oil past the compressor pistons. This has been remedied by placing a line condensate trap next to the pressure regulator. The overall functioning of the pneumatic system and controls has been very satisfactory, however.

Probably the biggest problem with which the plant has had to contend, and about which practically nothing can be done, is water condensation on both coated and uncoated pipe, floors, and metal equipment. Condensation on the pipe is to be expected, but when it occurs in the filter and on chemical feed room floors, it presents a constant cleaning problem.

Operation Records and Methods

Data in the first annual report indicate that the daily average pumpage of treated water to the mains was 3.0 mgd. As the Holland plant is designed for 14 mgd, and the smallest low-lift pump capacity is 5 mgd, it is apparent that the plant operates only 60 per cent of the time. The settling time is about five times the design rate. The prolonged settling time has not seemed to be as great an advantage as might have been expected. This is probably due to the low average turbidity of only 6 units. The Holland plant is well over volume demand at present, which is somewhat unique these days.

Chemical cost per million gallons is $2.47, based upon an average chlorine dose of 1.8 ppm and an alum dose of 6.9 ppm. Most of the time only prechlorination is used. This method of feeding chlorine was initiated because of difficulties caused by low pumpage. Although the plant at first maintained a finished-water residual of 0.4–0.5 ppm, there have been no complaints with residuals ranging up to 0.8 ppm. On the basis of these results, the mixed residual is now maintained at 1.0–1.3 ppm and the finished water residual is 0.6–0.8 ppm.

To double check the chemical feed and observe the type of floc formed, a mixed sample is collected by each shift in the laboratory and stirred at constant speed. Even at constant alum feed, the characteristics of the floc vary from day to day. The average alum dose for the past year has produced an average loss of head of 26 mil gal per acre per foot at a rate of 129 mgd. An analysis of costs reveals that an alum dose increase of 4 ppm would have to double the filter runs in order to be economically feasible.

There has been only one 7-day period during which odor has been a problem. The raw water had a strong, very definitely fishy odor after chlorination. Doubling the prechlorination dose did not alleviate the difficulty, and laboratory experiments showed that 6 ppm would be required. Without dechlorinating equipment, this procedure was not feasible. There was carbon on hand to remove the odor and this was fed to the rapid-mix flume with some emergency equipment. Carbon is always kept on hand in slurry form for such emergencies, and better equipment to meet these situations is being planned. Because these odor occurrences appear to be infrequent, use is made of the chemical feed trough by

simply moving a tank with a built-in stirrer over the trough with the lift truck that is used to handle the dry alum.

Public Relations

A clean, well landscaped plant is an important part of good public relations and considerable effort has been made toward this end in Holland, particularly because the plant is located in the middle of a recreational area and adjacent to a county park. The plant custodian pointed out before the landscaping was completed that the 1,200 pine trees that were planned as part of the planting would be catching debris for several years before they would grow from their original 18-in. size to become something of beauty. This part of the landscaping was therefore replaced by other trees. More lawn and a sprinkler system were also added.

The Holland plant's public relations got off to an excellent start with a public "Open House." Approximately 4,000 people went through the plant. Refreshments were served and each person was given a descriptive brochure on the plant and a booklet on water treatment. The local newspaper printed a special issue for the celebration. Since the opening, written invitations to visit the plant have been extended to the schools, youth organizations, and church groups. This program and the vastly improved quality of Holland's water supply have made a most favorable impression upon the public.

Operating Characteristics of Rapid Sand Filters

Herbert E. Hudson Jr.

A paper presented on Sep. 19, 1958, at the Ohio Section Meeting, Cleveland, Ohio, by Herbert E. Hudson Jr., Partner, Hazen & Sawyer, Detroit, Mich.

THE operating characteristics of rapid sand filters are dependent upon the work they are intended to do, their design, and how they are managed; what is expected of the filters depends on the standards of quality for the product served to the consumer. Three decades ago the water industry was content if treated water had a turbidity of less than 1 ppm, and the presence of gas formers in coliform-bacteria tests was tolerated provided the confirmed test was not often positive. Today filtered water with turbidities less than 0.2 ppm can readily be produced at all times, with an average of 0.05 ppm. Many plant operators are nowadays disturbed by a single presumptive test for coliform organisms and deeply concerned when a single confirmed test proves positive. The efficiency of rapid sand filtration must be given more attention if these modern standards are to be met.

Filter Bed Troubles

Although it was once believed that a filter had to be somewhat dirty in order to operate efficiently, it is now known that clean filters do a better job of treatment and that they remain in safe operating condition at times when dirty ones would fail (*1*). The condition of the filter bed is therefore an important consideration for the designer and the operator.

Filter bed troubles may be divided into two kinds: those that start at the top of the filter bed, and those that start at the bottom. Troubles that start at the top of the bed begin with compacted flocculated material, which combines with filter media to produce mudballs. The mudballs grow until they sink during the washing process and come to rest on the gravel bed. There they consolidate into clogged masses, which grow laterally and vertically until they often reach through the entire filter bed. These clogged masses cause maldistribution of backwash and displacement of gravel. Sometimes this displacement is almost volcanic, forcing the gravel upward completely through the filter bed and rendering it ineffective.

The accumulation of flocculated material within the filter bed also changes the filter media from incompressible to compressible and causes compaction during a filter run. This compacting is indicated both by sinking of the sand surface during the filter run and lateral shrinkage, which may open cracks in the bed and along the sidewalls. Through these cracks, water may penetrate to the underdrainage system, thus rendering the filter useless.

On the other hand, troubles may start at the bottom of the filter because of uneven wash water distribu-

tion. The maldistribution may be caused by faulty underdrains or by the gravel layers. There is a tendency for gravel to wander, even in the best filters, and thereby induce maldistribution of the backwash (*2*). This is not usually serious enough to impair the overall operation of the filter, but the effects may be cumulative and may ultimately result in projection of the gravel through the sand bed, thus causing failure of the filter. Gravel displacement also leads to leakage of sand into the underdrains and clear wells.

Washing Methods

The obvious remedy for filter troubles is to dismantle the filter, clean its components, and reassemble it properly. When done frequently enough, this can keep a filter in reasonably good condition. In some plants such rehabilitation is done as often as twice a year, which would usually be excessive; but in these plants it was necessary. In others, rebuilding may be necessary only once in 5–30 years.

Some plants have been able to eliminate mud ball troubles by more frequent washings. In one plant in which filter runs of 48 hr or more could be maintained, the operator washes the filters every 30 hr, in spite of the consequent head loss. This is good practice. Another effective measure is scouring the sand surface with a high-pressure stream once a month.

For troubles that begin at the top of the filter, preventive measures may often be taken. The high rate of backwash (20 gpm/sq ft, or more), formerly praised as a preventive measure, has not proved uniformly successful. The use of air wash, widely advocated in the past, has been increasingly abandoned in the United States, though they are still used successfully abroad. These methods probably fail because of a lack of vigor in the application of scouring force.

The most successful filter-washing method used in conjunction with backwashing is the surface wash. The Baylis fixed-jet system has about one ⅛-in. jet for each square foot of filter area. This system requires about 2 gpm/sq ft at approximately 50 psi. With its fairly extensive pipe network above the sand bed, it effectively cleans the sand.

The filter sweep system devised by Palmer uses fewer jets and requires about 0.5 gpm/sq ft under 50-psi pressure. This system uses jets in rotating pipe arms that swivel horizontally above the expanded bed and in the top of the expanded sand during backwashing. Although this system does not clean the corners as well as the fixed-jet system, it does a satisfactory job.

Both kinds of surface wash are subject to enlargement of the jet openings through the combined forces of scouring by sand and corrosion. Steel is therefore unsuitable for the jet; brass is better, but it also wears away. Plastic is reported to give good service.

Without the use of surface wash, mud balls may accumulate and form as much as 10 per cent of the volume of the top 6 in. of the bed in only a few months. Proper washing can prevent the formation of mudballs and thus eliminate all of the filter bed troubles that begin at the top of the bed (*1*).

Thus far this discussion has been intended to apply to all kinds of pretreatment plants, but softening plants present a further difficulty. In such plants there is the problem of cementation of sand and gravel by chemical precipitates. Owing to the deep pene-

tration of these materials, surface washing is less effective and backwashing is more important. Stabilization of the water before filtration by solids-contact reaction or by polyphosphate treatment also helps.

Underdrain Design

Troubles that start at the bottom of the filter, owing mainly to the maldistribution of backwash water, spring first from the design of underdrains. Underdrain types are given a thorough review by Hazen (*3*).

Through methods set forth by McNown (*4*), Fig. 1 was prepared for a perforated-pipe system in a filter 19 ft × 56 ft with underdrain laterals 9.5 ft long. The value Q_l/Q is the ratio of discharge through the orifice to the flow past the orifice in the lateral. This was plotted against a head-loss term (*4*), $\frac{h_f'}{V_l^2/2g}$, in which h_f' is the head loss through the lateral owing to friction, V_l is the average velocity through the lateral, and g is the acceleration due to gravity. The dashed portion of the curve represents the increased variation that would be expected if longer laterals were used.

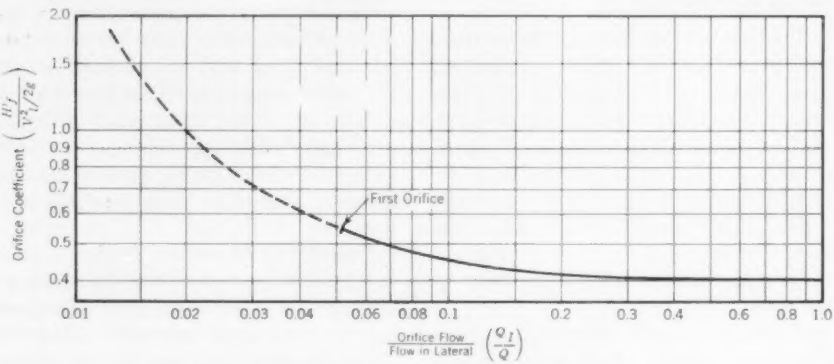

Fig. 1. Head Losses at Orifices in Lateral

The laterals of the filter from which data for this curve were taken were 9.5 ft long. The dashed portion of the curve represents losses that could be expected if the laterals were longer.

Figure 1 represents a hydraulic analysis of an underdrain system. It reveals that one generally ignored hydraulic characteristic of manifold systems is a vital consideration in the design of underdrains. The old practice of relating orifice openings to the area of the bed and similar methods do not produce adequate designs. In addition to the change in pressure along the lateral, a major design criterion is the variation in orifice coefficients along the manifold.

It can be seen from the figure that there is a tendency for the discharge through the orifices to be lower near the inlet to the lateral than near its terminus. This is because the water has a more difficult time turning to pass through the orifice when the lateral velocity is high. Consequently the orifice "contractions" are more severe at high flow velocities.

There is a substantial, systematic variation in the flow through the orifices along each lateral. The variation is greater for systems having a greater number of ports, or orifices. On the other hand, the openings into the filter bed must be spaced closely enough to insure adequate dispersion of wash water throughout the gravel. Once the geometry of any manifold has been determined, design curves such as that in Fig. 1 may be prepared.

One method of improving underdrains is to use a larger number of small-diameter orifices in the laterals. This invariably results in greater head loss, but it improves the distribution of wash water. Loss of head is not a serious problem, because the amount of water used in filter washing should usually be less than 2 per cent of the total production in a well operated plant. There is, of course, no reason to waste power that could be saved through proper design.

Gravel

The gravel layer cannot be expected to operate efficiently if the underdrain system is poorly designed. The gravel, during backwashing, serves only to distribute the water from each orifice through the area served by that orifice. It is good to have as many as five layers of gravel, each several inches thick, so that variations in level during installation will not be too important, and so that the gravel will aid in distributing the wash water throughout the filter area. The layers of gravel should be so graded that the sizes vary only gradually from one level to the next. A typical set of sizes is: top layer, 10–8 mesh; second layer, 8 mesh–$\frac{1}{4}$ in.; third layer, $\frac{1}{4}$–$\frac{1}{2}$ in.; layer, $\frac{1}{2}$–1 in., and so on. The top layer must be fine enough to support the filter sand design during filtration. For a sand having an average size of 0.7 mm, it has been customary to require that the top layer of gravel be 10–8 mesh.

Although it was formerly believed that the top layer should be about 4 in. thick, in order that some would still remain even if part of the layer moved during washing, it has been found that a thinner layer serves equally well. Studies of building underdrainage and gravel packs for wells (*5*), indicate that there is no significant intermixing of adjacent, nonuniform layers if the median sizes of the layers differ by a factor of no more than five. It is assumed that this factor would be reduced somewhat for relatively uniform materials and that the 10–8-mesh gravel could be replaced by 8–6-mesh material with the filter sands that are now generally used.

The most recent work on the problem of gravel movement during backwashing has been done at Chicago. Experimental work on higher-density materials (normal silica gravel has a specific gravity of 2.65) is being carried out. Higher-density materials are available with specific gravities in excess of 3.00.

Filter Inspection

Routine inspection of filters should be standard practice. The inspection measures should include examination of each filter at least once a month and sounding during backwashing to see whether movement of gravel has occurred. Figure 2 is an illustration of a typical filter bed showing in detail the variation in gravel levels found by probing through the sand during backwashing with a $\frac{3}{4}$-in. wooden rod (*6*). Variations in this filter are normal and

may be expected in many filters. Variations greater than these may be cause for concern. It is very easy to feel the top of the gravel layer with such a rod. The rod should be so graduated that variations in distance from the water surface to the gravel surface may be measured rapidly.

A second measure for inspection of filters is sampling for mudballs. This is done by taking a sample of known volume from the top 6 in. of the bed, and screening it under water through a 10-mesh sieve. Any material that remains on the sieve (exclusive of gravel) is classified as mudball material. The percentage of mudballs by volume should not exceed 1 per cent.

An important part in the inspection of filters is observation of what occurs during backwashing in each bed. If the sand boils up in a particular filter, always at the same location, sounding will probably reveal disruption of gravel beneath the boil. Inspection of the filters may also reveal clogged masses that project through the filter bed and are evident at the surface as well as cracks in the bed that occur during filtration. The detection of the cracks usually requires drainage of the filter before backwashing. The sounding pole also reveals clogged masses, which give a soft, passive resistance instead of the "chinky" feel that gravel has.

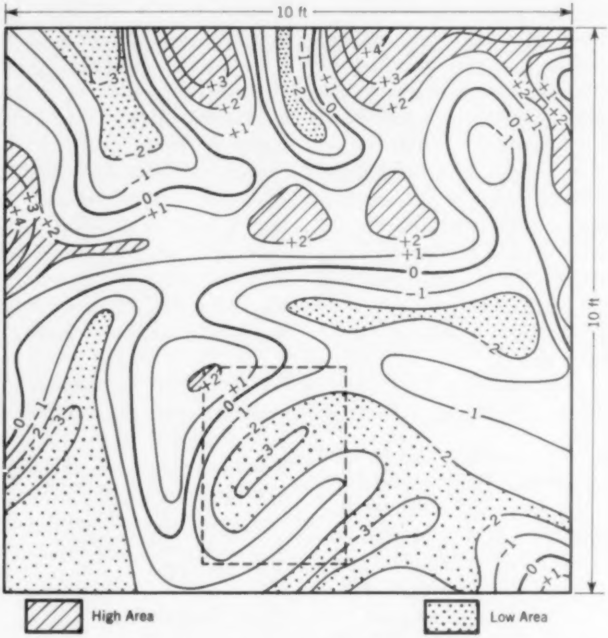

Fig. 2. Contours of Gravel Surface in Typical Filter

The data upon which this diagram is based were obtained by probing the filter with a sounding pole during backwashing. The contours are expressed in inches.

Another way to keep track of filter bed condition is continuous monitoring of the filtered-water quality from each filter. An easy way to do this is to provide a cotton-plug unit for each filter, and to operate it at such a rate that the plug will yield good values when changed weekly. A rate of 40 ml/min should be satisfactory for turbidities up to 0.3 ppm. The cotton plugs, weighing 2 g, are ashed and the residue is weighed. The weight of residue divided by the amount of water that has been passed through the cotton plug (which is controlled by a simple float valve—orifice arrangement) gives the quantity of suspended material appearing in the filter effluent in milligrams per liter. Normally all filters should produce approximately the same amount of suspended matter in the filter effluent. Any abnormally high value is cause for inspection. It may be found that the sand in the unit is of different size than in the other filters, or that there are clogged masses, disturbed gravel, cracking, or pulling away from the sidewalls.

Filter Media

An important fact about filter media is that the quality of filtered water is dependent upon the size of openings through the bed. The larger the openings the less clear the effluent will be. In a plant where sanitary hazards are low and water has been well treated before filtration, coarse sand may safely be used. If pollution hazards are high, however, and pretreatment facilities are not completely satisfactory, finer sand is required. Sand finer than 0.5 mm is rarely used, because coarser sand will generally produce safe water with turbidities of less than 0.2 ppm, and finer material causes great trouble in short filter runs. The lengths of filter runs are directly proportional to the square of the sand size, and very fine sand may cause rapid clogging.

Most filter sands are rounded, and there is clear evidence that the rounded particles produce clearer water than angular ones. This is because the angular materials give greater porosity. It has been found that, for a given size, the porosity of rounded sands is about 43 per cent, whereas the porosity of angular sands ranges from 48 to 55 per cent.

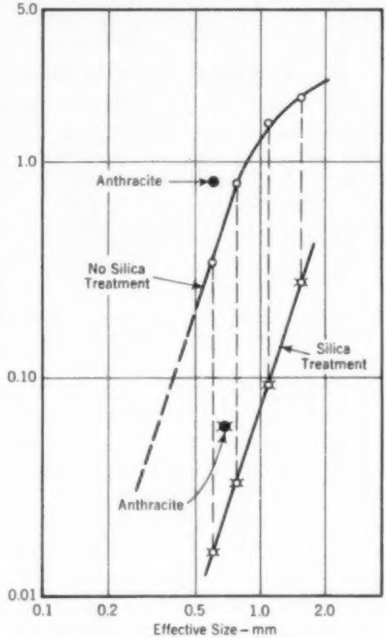

Fig. 3. Turbidity Removal With Various Filtering Materials

Turbidity was determined by cotton plug filter. The curve for treatment without activated silica shows that effluent turbidity from coarse materials approached that of settled water.

Figure 3 is based on data collected at Chicago (*7*). From the curves it

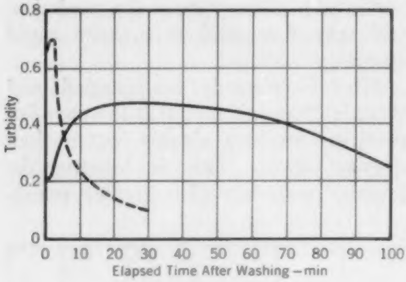

Fig. 4. Effluent Turbidity After Filter Washing

The rise in turbidity just after the beginning of a filter run is due to breakthrough of flocculated material broken up during the washing process. This turbidity cleared up much more rapidly with 0.5-mm sand (dashed curve) than with 0.9-mm sand (solid curve).

can be seen that filtered-water turbidity is directly proportional to the cube of the sand size. The figure also demonstrates the difference in filter effluent quality for two flocculation conditions. Flocculation conditions are more favorable during warm weather than cold, and lower turbidities are easily obtained. For both conditions it will be noted that the turbidity of the effluent from the bed of angular material was more than twice that of the effluent from the rounded-sand filter.

These results do not condemn the angular materials, which may safely be used, if a corresponding size reduction is made, with equal results. For filters of equal clarifying ability, filter runs will be approximately equal in effectiveness, regardless of porosity (*8*).

The use of anthracite (an angular material) has grown in recent years. Its use may sometimes be accompanied by a lowering of filtered-water quality. Anthracite filtering materials are lighter than sand and therefore more subject to troubles that start at the top of the filter bed. On the contrary, they are less subject to the troubles that start at the bottom of the filter bed, because coal is much lighter in weight than gravel and there is little tendency for the gravel to move.

There is a very useful place for the light-weight angular materials as an additive to the sand bed. Several inches of anthracite of 1-mm effective size may be placed on top of a filter having a sand of 0.5 mm effective size. Under backwashing alone, the materials stay completely separate. There is some mixing when surface washes are used, but surface washing is necessary in order to offset the increased mudball-forming tendencies of the lighter-weight materials. Restratifi-

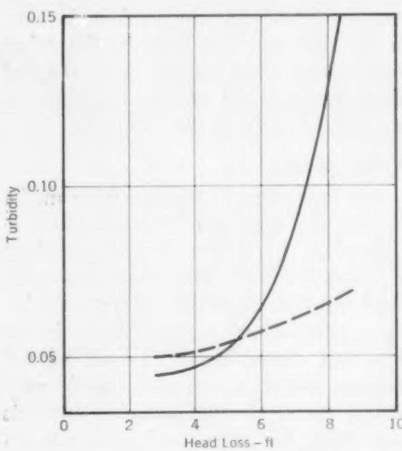

Fig. 5. Breakthrough at End of Filter Run

The solid curve represents the increase in turbidity with head loss with weak floc; the dashed curve represents the increase with normal floc. Turbidity data were gathered with a microphotometer.

cation may be obtained by a period of low-velocity upflow after the surface wash is turned off. The use of coarse anthracite on top of sand substantially lengthens filter runs.

Breakthroughs in Filters

At the start of a filter run, there is a minor breakthrough of suspended matter into the effluent. It used to be the custom to operate filters to waste at the start of the run for a period long enough to flush away this breakthrough, which may last for as much as 1½ hr, but which is usually complete in 30 min. The startup breakthrough is probably caused by floc within the sand bed that is broken up during the washing process. The breakthrough occurs whether surface wash is used or not. Figure 4 shows breakthrough curves for two filters having fine and coarse sand (0.5- and 0.9-mm effective sizes, respectively). Both filter beds were clean and in good condition. There is reason to believe that the startup breakthrough generally lasts longer with coarse materials than

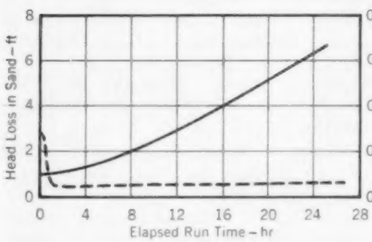

Fig. 6. Head Loss and Turbidity Removal With Strong Floc

Comparison of the head loss (solid) curve with the turbidity (dashed) curve shows that there is a fairly constant rate of turbidity removal when the clogging rate increases throughout the filter run. Breakthroughs rarely occur under these circumstances.

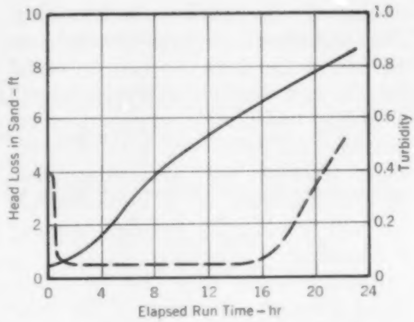

Fig. 7. Head Loss and Turbidity Removal With Weak Floc

The solid curve represents head loss; the dashed curve represents turbidity. The sudden rise in turbidity indicates a breakthrough, and is caused by the inability of the weak floc to remain within the filter bed. As can be seen, the breakthrough was preceded by a decline in the clogging rate.

with fine ones. The duration of the breakthrough for the coarse material is greater than can be accounted for by the 30-min supply of water held above the sand within the filter, and is evidence that the coarse material requires considerable time before it functions with full effectiveness. This period may last as long as 5–10 per cent of the total filter run.

Toward the end of filter runs, especially when flocculation is not strong, there may again be filter breakthroughs. An example of this is shown in Fig. 5 for a period of weak flocculation, as compared to a period of moderately strong flocculation. The data were obtained by using a continuous-recording microphotometer. It can be seen that in both cases there was a tendency for the turbidity to rise at the end of the filter runs, but that this tendency was much greater when floc was weak. This difficulty may be

eliminated by earlier backwashing. The occurrence of breakthroughs at the end of the filter run may be found only by spot-checking of filters, or with recording turbidimeters, which are expensive. A "sight" well in the filtered-water reservoir may give information on the existence of a breakthrough, but will not identify the filter causing the trouble.

Breakthroughs may also be anticipated from the behavior of the relationship between time and head loss. For a constant-rate filter, it is usual for the loss of head to be virtually constant at the beginning of the filter run, and then to rise at an increasing rate throughout the filter run. This is shown in Fig. 6. Under such conditions, breakthroughs are uncommon. The shape of a filter run curve is difficult to observe from the circular charts usually used. Strip charts are far more informative. As breakthrough threats increase, the filter run curve tends to become straighter, and when it is almost completely straight, the hazard is substantial. The worst condition of all is that shown in Fig. 7, which is a severe breakthrough, in which the floc is not strong enough to be held within the filter beds, and the loss of head increases less rapidly toward the end of the filter run. Under conditions such as this, filtered-water turbidities may be nearly as high as the applied water turbidities. Data for Fig. 6 and 7 were obtained with a recording microphotometer.

Summary

Filters should be so operated as to produce filtered water turbidities of 0.2 ppm or less. They are hampered in this aim by becoming fouled with mudballs and the troubles that grow out of mudballs. They are also hampered by inadequate wash water distribution, which may be due to disturbance of the gravel or improper underdrain design. Surface washing prevents troubles that start from the top of the bed.

Coarser sands may be safely used where there is good pretreatment, but it must be remembered that the concentration of suspended matter in the effluent is proportional to the cube of the effective size. Angular materials, such as anthracite, must be used with discretion; the adverse effects of their high porosity necessitate the use of finer materials to obtain results equal to those produced by rounded materials.

Filter breakthroughs can ruin filtered-water quality. They can be detected by frequent checking of turbidity. Analysis of filter run curves is useful in guarding against the occurrence of breakthroughs.

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Importance of Underdrains in Filter Backwashing

Raymond E. Hebert

A paper presented on Apr. 24, 1958, at the Annual Conference, Dallas, Tex., by Raymond E. Hebert, Pres., Filtration Equipment Corp., Rochester, N.Y.

FOR the purposes of this discussion only the backwash cycle of rapid sand filters will be considered. In order to evaluate an underdrain system, its relation to the overall problem of filter operation should be considered. Although the design of an underdrain system is important, satisfactory results cannot be obtained unless means are provided to stabilize the sand and gravel areas, remove a large percentage of foreign matter from the sand, and reduce the use of backwash water to a minimum.

The backwash operation can be divided among these three separate areas of the filter, each utilizing a different principle of hydraulics: [1] the distribution area from the bottom of the filter through the underdrain and large gravel up to the bottom of the small gravel layer; [2] the filtering area, which includes the small gravel and sand; [3] the cleaning area from the level of the expanded sand to the level of the overflow into the wash troughs. Figure 1 indicates the location of these three areas as well as their relative resistances to flow in a typical filter under the conditions noted.

Distribution Area

The purpose of the distribution area is to spread the water so that it reaches the bottom of the filtering area at a level plane. In order to accomplish this, the design should allow for free movement of the water in all directions so that it can seek its own level. All of the tests which have been conducted indicate that a satisfactory distribution can be secured with a free-flowing underdrain system consisting of noncorrosive plates with 1-in. diameter holes on 6-in. centers and baffles to prevent the gravel from plugging the holes. This type of underdrain offers very little resistance, and it is therefore unnecessary to secure it to the filter bottom. It not only reduces the cost of the original installation, but also has the advantage of being removable and replaceable at very little labor cost if it should be necessary to clean the filter. The distance between the bottom of the filter and the bottom of the underdrain can be controlled by the length of the supporting legs.

This system provides stability and a very even distribution when the backwash water is introduced under the filter bottoms in excess of 25 gpm/sq ft—even before any gravel is put in place. Although tests do not indicate a maximum, it is recommended that the backwash intake be designed to maintain a velocity of ap-

proximately 2 fps at the point where the water enters the underdrain area.

A careful investigation of the determination of the orifice ratio formula (ratio of orifice area to total bed area), used to calculate size of orifices in lateral systems, indicates that the calculations are based on orifice discharge into the atmosphere. Because the orifices in an underdrain system discharge into water in the bottom of a tank with a head of 5–8 ft, this formula does not apply.

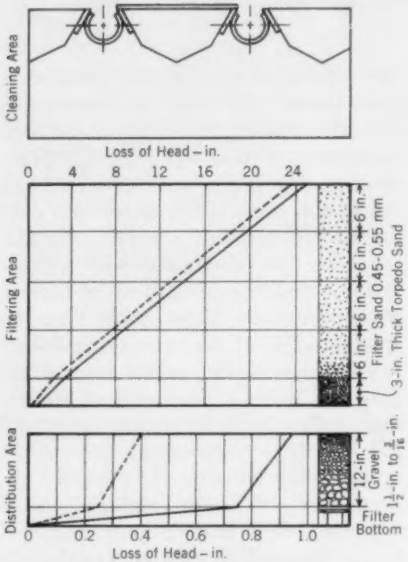

Fig. 1. Section of a Rapid Sand Filter

The graphs represent loss of head at successive levels in the filter bed. The dotted pair represents backwashing at 15 gpm to give a 24-in./min rate of rise; the solid pair represents backwashing at 22.5 gpm to give a 36-in./min rate of rise.

Many present underdrain systems are designed on the basis of this formula, and although their performance is satisfactory, in the evaluation of their relative merits serious consideration should be given to the wasteful and unnecessary cost of pumping against this high resistance. There are some instances where the small-diameter holes have corroded or become plugged, creating a resistance so great that it is impossible to supply sufficient backwash water for proper cleaning.

All of the tests indicate that a lateral system is not necessary for proper distribution of the backwash water, and that the underdrain system should be designed to support the gravel and sand with as little resistance as possible. As shown in Fig. 1, the largest percentage of the total resistance through the filter is in the small-gravel and sand area, which takes over control of the flow as soon as the water enters that section of the filter. It is therefore obvious that the safety factor in the distribution area is extremely large.

Filtering Area

As soon as the water reaches the filtering area, its flow is controlled by the resistances of this medium in the various areas of the filter, and the greatest flow is through the areas of least resistance.

Observations of the operation of a 144-sq in. test filter at the Rochester, N.Y., municipal plant, showed that the gravel remains stable at a backwash rate as high as 30 gpm/sq ft (48-in. rise). This would seem to indicate that movement of the small gravel in a large filter is caused by lateral flow of the water with increased velocity adjacent to areas of low resistance. This horizontal velocity can become very high and is increased each time the filter is backwashed. The movement of the small gravel into the sand

layer causes areas of greater differential in resistance.

In order to prevent horizontal movement of the small gravel in working installations, by duplicating as nearly as possible the 144-sq in. filter in which these tests were conducted, baffles have been fabricated 21 in. high in an egg crate effect, forming 12-in. and 24-in. squares. These extend from the top of the large gravel through the balance of the gravel up into the sand layer. An installation has been made at the Marathon Corp. at Oswego, N.Y., and at the Belmont plant in Philadelphia. A new 24-in. square test filter with transparent sides is being constructed to make it possible to determine the maximum dimensions of the area formed by the baffles as well as the most effective height required.

Data are also being collected on the cause and effect of air accumulation under filter beds.

Cleaning Area

After the water has passed through the sand area, its primary function is to transport the foreign matter that has been separated from the sand into the wash troughs. As the solids-carrying capacity of water is proportional to its velocity, this area should be designed to maintain or increase the velocity. By the introduction of a baffle from just above the level of the expanded sand to the lip of the overflow on the test filter, it was observed that the higher velocity produced a much better and faster movement to the overflow. Because the flow was directed, it also eliminated the "short circuiting" of the heavier particles. Preliminary calculations indicated that savings of more than half of the backwash time at the same flow rate and a much higher percentage of cleaning could be obtained by this method.

In order to secure accurate data on a filter in actual operation, sectional baffles, formed like the hull of a boat, were designed and installed in one of the filters at Marathon Corp. Data are being compiled on this installation and comparison with the other five filters at this plant should give an indication of its practicability.

Inspection of a number of filters indicates that a large percentage of the difficulty in operation is caused by not removing a high enough percentage of foreign matter by backwashing. This condition allows the material that is left in the filter to produce uneven resistance in the sand bed, inefficient filtering, and, eventually, an upset filter bed.

Conclusion

Because of the unanswered problems brought out in this discussion, and after a review of the available, published information on this subject, the author feels that continued research should be promoted. This should not only include methods for the design of new plants, but it should also be extended to promote more efficient operation of the thousands of rapid sand filters that are furnishing a large percentage of municipal water supplies, and are, in many cases, struggling to meet the increasing demand.

Nature and Effects of Filter Backwashing

John R. Baylis

A paper presented on Apr. 24, 1958, at the Annual Conference, Dallas, Tex., by John R. Baylis, Engr. of Water Purification, Dept. of Water & Sewers, Chicago, Ill.

FILTER bed trouble is as old as rapid sand filtration. Some of the causes of filter bed trouble have been corrected, but left unsolved are the major causes, such as movement of the fine gravel during backwashing, forming mounds, and leakage of sand through the beds. The formation of troublesome, mud-clogged areas in filter beds is now being largely eliminated through use of surface wash systems. When the surface wash was developed into a practical aid in backwashing, it was thought that most of the filter bed troubles would be at an end. Clogging trouble has been eliminated, but it is not known how to construct filters in which the gravel will remain in place indefinitely.

The fine gravel in filter beds moves from its original place in the bed during backwashing, forms mounds, and becomes mixed with the sand. The movement of the gravel is such that in places the sand comes in contact with gravel coarse enough to allow it to flow gradually downward through the gravel and eventually pass out with the filtered water (Fig. 1). Backwashing of rapid sand filters is a necessity. The period between backwashings varies from a few hours to 2–3 days. Filters are designed with a view to backwashing as soon as the bed becomes so clogged with coagulated material that the desired rate of filtration cannot be maintained.

Various Opinions

Ideas vary widely as to the rate at which a filter bed should be backwashed. Some filter operators are of the opinion that the backwashing should be rapid enough to expand the bed about 50 per cent. Others believe the rate should be lower. The author is inclined to think that better friction between the sand grains, to loosen the coagulated material adhering to them, is accomplished at about 20–25 per cent expansion of the sand bed, and that this is a better rate to use than 50 per cent expansion. It at least allows better action of the surface wash.

Various opinions have been advanced as to what causes the top gravel to move about in the bed, to the extent that it mixes with the sand and causes uneven distribution of the backwash water. Some have expressed the view that the trouble is the result of uneven distribution of the backwash. This, of course, can be a contributing factor, but it is not the main cause in many filtration plants. Filters with perfect water distribution under the beds often get out of order. The author, some years ago, made a study of the cause of mixing of sand and gravel at their junction (*1*). A study

was also made to determine the cause of clogged areas. This is a problem that no type of filter bottom, except porous plates, can solve. But while solving one trouble, the porous plate creates other difficulties which have prevented its widespread use. Anyone who hopes to prevent gravel from mounding in filter beds by using some improved design of underdrain can only look forward to disillusionment, and should, therefore, seek some other solution.

Lucky is the plant that does not have to rebuild filter beds more often than once in 20 years, but there are a number of plants where the beds have been in use longer than 10 years with no great amount of trouble. Filter beds in a few plants have to be rebuilt almost every year. The author once worked in a plant with 32 filters where all the beds had to be rebuilt three times in 10 years.

When the fine gravel in some of the large filters in the South District Filtration Plant mounded to the extent that some of them had sand leakage in less than 10 years of use, another study of gravel movement and its causes was undertaken.

Two types of experimental filters were used in the backwashing tests. One was a glass-walled filter, 9 sq ft in area, and the other was a clear-wall, 6-in. plastic pipe. Figure 2 is photograph of the plastic filter as set up. The plastic filter was constructed about 2 years ago. The first experiments on the 6-in. pipe filter were to determine the backwash rates required for particular gravel bed expansions.

Table 1 gives tests on backwashing beds of gravel in the pipe filter, 6.54 in. inside diameter. The relation of backwash rate to loss of head is plotted on the graph in Fig. 3. It is interesting to note that the head loss does not increase materially above 3 ft for the various sizes of gravel. The backwash rate needed to produce partial suspension of the gravel, of course, increases rapidly with the increase in size of gravel.

Jet Action

From these tests it is evident that high water velocities are required in order to expand gravel beds. The rates are higher than would ever be used in rapid sand filter washing. It is the jet action of the water mixed with sand that produces velocities which will move particles of gravel. To illustrate, if a backv ,sh rate of 30 gpm/sq ft, or 48-ipm vertical rise, is required barely to move gravel of finer size, then it is evident that jet action must need greater velocities. That this

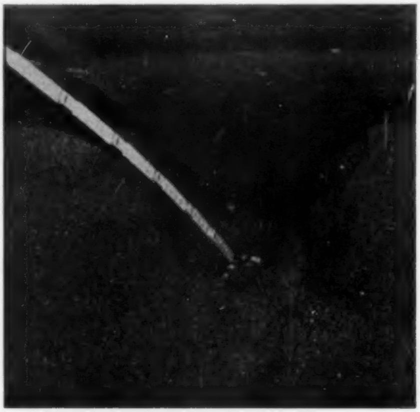

Fig. 1. Hole in Sand Caused by Mixing of Sand With Gravel

This hole, which extends all the way through to the gravel layer, is a typical example of the damage caused by sand leakage.

is true of sand bed expansion of more than about 8 per cent is evident from approximate measurements of the velocity of the jet. A velocity of 1 fps is 720 ipm. This is twelve times the velocity required to start suspension of $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel. Water that is mixed with a considerable amount of sand, as it occurs in the beds, will have a greater lifting force than water with no mixture of sand. In order that sand and water continue to mix, the sand must be moving constantly into the places where sand jet actions

TABLE 1

Backwash Characteristics of Gravel

Gravel Expansion *in.*	Head Loss *ft*	Backwash Rate *gpm/sq ft*	
		First	Second
$\frac{1}{12}$–$\frac{1}{8}$-in. gravel			
0.0	0.1	1.68	2.01
0.0	0.2	3.00	5.55
0.0	0.3	4.20	5.97
0.0	0.6	7.51	11.00
0.0	0.9	10.73	15.02
0.0	1.2	13.41	18.98
0.0	1.5	15.69	22.17
0.0	1.8	18.50	25.30
0.0	2.1	21.46	28.52
0.0	2.4	24.14	31.47
0.0	2.7	26.82	34.87
6.0	3.0	57.67	50.43
12.0	3.05	82.88	86.37
$\frac{1}{8}$–$\frac{1}{4}$-in. gravel			
0.0	0.1	3.94	5.58
0.0	0.2	6.44	9.10
0.0	0.3	9.40	12.75
0.0	0.6	15.28	20.43
0.0	0.9	20.26	27.73
0.0	1.2	24.99	33.39
0.0	1.5	29.61	39.44
0.0	1.8	32.58	44.42
0.0	2.1	36.35	48.28
0.0	2.4	40.13	53.13
0.0	2.7	43.43	57.68
6.0	3.0	92.27	104.29
12.0	3.08	123.86	133.05
$\frac{1}{4}$–$\frac{1}{2}$-in. gravel			
0.0	0.1	7.64	9.95
0.0	0.2	11.81	16.75
0.0	0.3	15.15	22.30
0.0	0.6	24.50	34.50
0.0	0.9	31.00	44.0
0.0	1.2	36.30	53.4
0.0	1.5	42.8	61.2
0.0	1.8	47.3	67.6
0.0	2.1	52.3	72.7
0.0	2.4	56.8	69.7
0.0	2.7	60.7	85.2
0.5	3.0	65.3	93.4
6.0	3.0	138.6	157.4
12.0	3.3	181.5	211.0
18.0	3.5		242.5
25% $\frac{1}{8}$–$\frac{1}{4}$-in., 25% $\frac{1}{12}$–$\frac{1}{8}$-in., 50% sand			
0.0	0.1	0.18	0.70
0.0	0.2	0.32	0.97
0.0	0.3	0.54	1.40
0.0	0.6	0.92	2.47
0.0	0.9	1.29	3.58
0.0	1.2	1.61	4.70
0.0	1.5	2.58	6.03
0.0	1.8	3.22	9.93
0.0	2.1	2.96	12.89
0.0	2.4	3.35	17.98
0.0	2.7	3.99	21.70
0.5	3.0	5.23	23.85
0.5	3.3	5.92	28.25
3.0	3.0	15.53	
6.0	3.05	20.35	
9.0	3.05	25.00	32.55
12.00	3.00	28.95	

occcur; otherwise there will be jets without sand. Although the amount of sand in the jets may be much lower than in other parts of the expanded bed, there is enough for the mixture to have a specific gravity greater than unity.

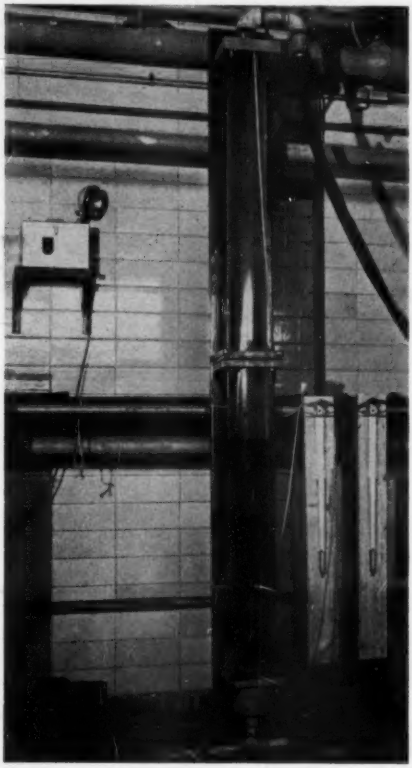

Fig. 2. Six-inch Plastic Pipe Filter

The filter as set up for the experiments. The floodlight to the left of the filter was used in making most of the photographs used.

Some idea of the unusual force of jet action may be obtained by observing it in transparent-wall filters. Rather than passing evenly from the gravel into the sand over the gravel surface, the water passes from the unmoving gravel quite rapidly in places, and slowly in others. A filter washed at a 25 per cent bed expansion may have jets of water that rush upward from the material not in motion into the expanded material at rates of more than 8 ips. Attempts to measure the velocity of the jets years ago indicated velocities as high as 0.8–1.1 fps. Recently, means have been devised for photographing sand jet action, and this also has indicated velocities in the jets of about 1 fps. An unusually long exposure is made. The exposure may be approximately 0.2–0.5 sec, instead of the usual 0.02 sec or less for a flash-bulb. With low backwash rates the exposure should be about 0.5 sec to show the streaking action of sand jets. For higher backwash rates, the exposure time should be less to show the spots where there is the greatest velocity.

Gravel Grading

The bed of the filter first used in a series of tests was a duplicate of the large (1,400 sq ft) plant filters in regard to sand and gravel depths and sizes. In Fig. 4 the lower part of the filter is shown after construction and before backwashing experiments were started. The bed consists of 25.5 in. of sand and approximately 12 in. gravel. The three top layers of gravel are of the same depth as the layers in the large plant filters. The next lower layer also is of approximately the same thickness as the layers in the plant filters. Photographs and motion pictures of the backwashing did not tell the full story because they did not show where jet action was taking place. An attempt to record the mo-

tion by placing plastic around the filter and marking the jet paths with a colored pencil failed.

Photographs

In experiments with various exposure times, it was found better to use floodlights instead of flashbulbs, and give 0.2–0.5-sec exposure, depending on the sand jet velocity. This shows the sand grains in rapid motion in the form of a streak. Where the grains are shown clearly and rounded, it indicates almost no motion. This is illustrated by the two photographs in Fig. 5. There is nothing to illustrate where the sand jets are taking place in the photograph on the left, but they do show in the one on the right, although the sand grains over the filter are not plainly seen. The main purpose is to show where there is rapid motion of the sand and the direction it is traveling. This is reasonably clear in the photograph on the right.

Figure 6 shows three photographs of a 4-in. sand expansion and illustrates the advantage of the new method. The backwash rate to produce this sand expansion was 13.90 gpm/sq ft. In Fig. 6 there are several places above the gravel where sand grains are plainly visible. In these places the grains made no appreciable movement during the 0.5-sec exposure. An interesting part of this picture is the increase in height of the gravel surface it shows. That the filter may have been tilted slightly must be taken into account.

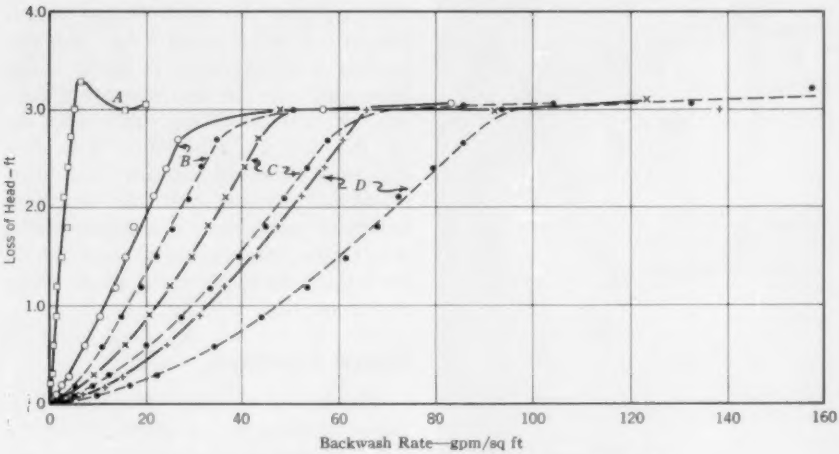

Fig. 3. Loss of Head in Backwashing Gravel Beds

Each gravel bed washed backwashed twice to show that when moved in backwashing, gravel settles back in a more porous condition. Solid curves represent first backwashing; dashed curves, second backwashing; Curve A, *mixed sand and gravel; Pair* B, $\frac{1}{12}$–$\frac{1}{8}$ *in.; Pair* C, $\frac{1}{8}$–$\frac{1}{4}$ *in.; Pair* D, $\frac{1}{4}$–$\frac{1}{2}$ *in.*

Gravel Height Increases

Figure 7 shows the filter being backwashed at a rate to produce 4-in. bed expansion. The high mounding of the gravel should be noted. Original level

was at the height of a small pipe tap through the plastic tube for attachment to a gage. This pipe connection is visible in the center illustration. The gravel level is considerably above the original level in all three photographs. Measurements around the tube perimeter show that the gravel level averages about 3 in. above its original height.

The increase in gravel volume is interesting because there appears to be little visible mixing of the sand and gravel. One has to realize, of course, that only the perimeter of the sand gravel bed is visible and little of what has taken place near the center is known. It would be difficult, however, to believe that enough gravel was moved from the center to account for the full increase in gravel height.

At 6-in. sand expansion, the filter was backwashed at a rate of 20.65 gpm/sq ft. The sand stood about an inch above the original level after the backwash valve was closed.

When the filter was backwashed at a rate of 25.15 gpm, it produced a 9-in. bed expansion. The gravel was not greatly off level. The sand jets extended higher, with one about 15 in. above the gravel.

Backwashing at a rate of 31.90 gpm/sq ft expanded the bed 12 in. The gravel level for material not in motion was close to the original level as shown in Fig. 8. A considerable portion of the small gravel was expanded with the sand, and with a slow cutoff of the backwash valve it settled back to produce a filter bed with the gravel above its original height. The highest sand jets extended about 15 in. above the stationary gravel.

Backwashing at a rate of 36.55 gpm/sq ft gave a 15-in. bed expansion. The gravel again became mounded when there there was disturbance of the larger gravel. Most of the finer gravel was mixed with the expanding sand. There was rapid motion of the sand and water in places, and the sand jets extended upward 15–18 in. above the gravel.

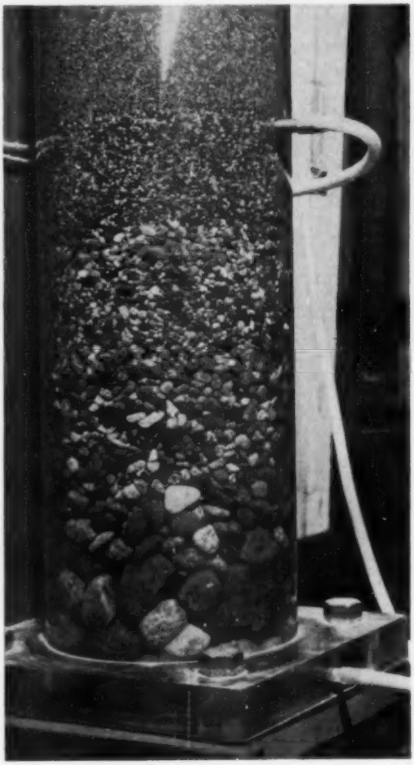

Fig. 4. Gravel Layers in Plastic Pipe Filter

This photograph shows the gravel layers in the filter as constructed, before any expansion of the bed.

Test Results

1. There is sand jet action at any expansion of the bed. The jets in-

crease in velocity and height as the backwash rate is increased.

2. There is movement of gravel by the sand jets at all expansions of the bed over 2 in., or about 8 per cent. Mounding is caused by pickup of gravel in a sand jet and its deposit in places where there is no rapid, upward flow of the water.

Gravel size—*in.*	Depth—*in.*
$1\frac{1}{2}$–$\frac{3}{4}$	6
$\frac{3}{4}$–$\frac{1}{2}$	4
$\frac{1}{2}$–$\frac{1}{4}$	$3\frac{1}{2}$
$\frac{1}{4}$–$\frac{1}{8}$	1
$\frac{1}{8}$–$\frac{1}{12}$	1
Original sand	$24\frac{1}{2}$

The main difference in the gravel bed construction was the substitution of an inch layer of gravel for the two

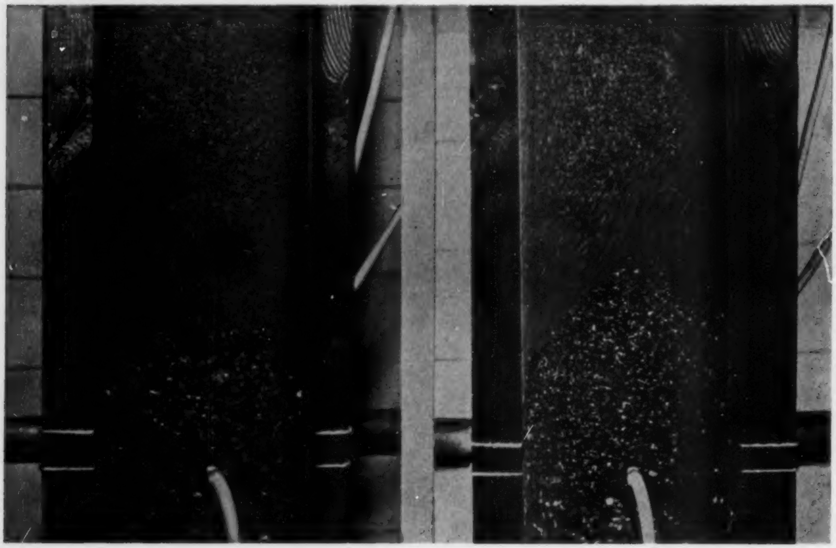

Fig. 5. Comparison of Flashbulb and Floodlight Photographs of Filter Backwashing

Left photograph (flashbulb) shows no motion of the sand, whereas the streaming is clearly visible in right photograph taken with a floodlight.

3. In the filter used, the gravel mounds were as high at a 4-in. bed expansion as they were at any greater rate of backwashing. This condition may or may not be true for larger filters.

Thin Gravel Layers

On Sep. 25, 1957, the 6-in. plastic pipe filter was rebuilt as follows:

upper layers, instead of $2\frac{1}{2}$ in. for the top layer and $3\frac{1}{2}$ in. for the second layer. The two bottom layers of coarse gravel were made deeper so that the top of the gravel would extend to the small pipe takeoff.

Backwashing at a rate of 9.35 gpm/sq ft gave a 2-in. sand expansion; there was very little sand jet action at the boundary between the sand and gravel. In places above the gravel

layer, the sand moved rapidly enough to be detected. After a 24-hr backwashing at this rate, there was no observed movement of the gravel.

Backwashing at a rate of 15.65 gpm/sq ft gave a 4-in. sand expansion. Very little sand jet action was noticed at this rate. There was slight streaking on the photographs of the upper part of the filter bed, which indicated more rapid sand motion. The gravel layer, however, was moved somewhat, and piled up about an inch above its original level along the side of the small pipe tap.

The filter was backwashed for 4 days at a rate intended to expand the bed 4 in. One well defined jet was developed and extended 8–9 in. above the gravel surface. The gravel was mounded about 2 in. above the original level in places. No depression appeared in the gravel surface, and it may be that the gravel was moved from the center of the filter over against the wall.

At a backwash rate of 20 gpm/sq ft to give a 6-in. bed expansion, a sand jet was distinctly visible. There was some mounding of the gravel—up to

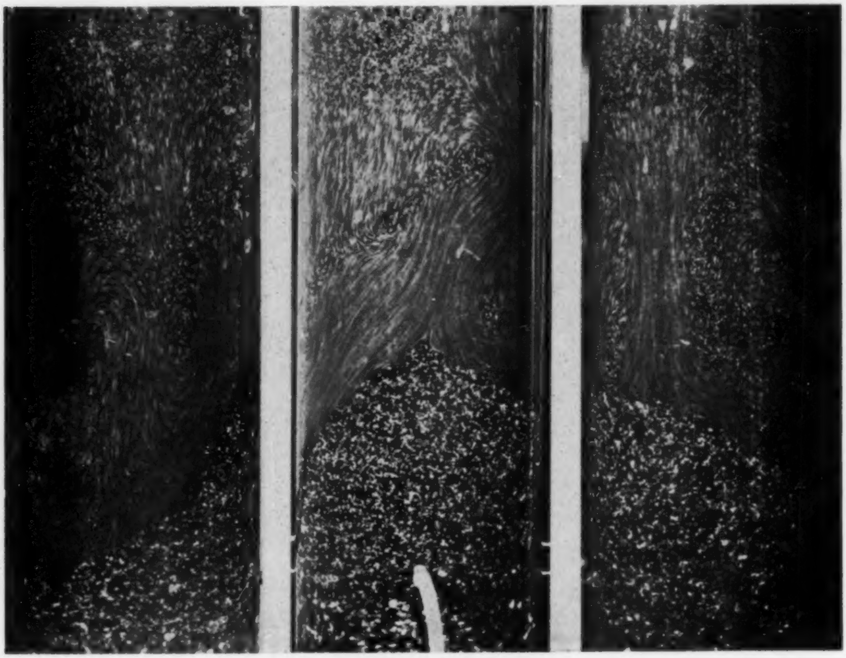

Fig. 6. Bed Expansion of 4 in.

Exposure time for these photographs was 0.5 sec. The backwash rate to produce this expansion was 13.90 gpm/sq ft. Note the increase in height of the gravel level over the original level.

about 3 in. above the original level. At no place around the periphery of the filter had the gravel been moved to where it was lower than the original depth. Either there was expansion of the gravel or a considerable amount of gravel was pulled from the center of the filter and mounded at the side. jet extended upward more than 6 in., although the sand was moving quite rapidly 12–20 in. above the gravel surface. There may have been an interior sand jet below that level that did not show against the filter.

The backwash rate required to produce a 10-in. bed expansion was 29.40

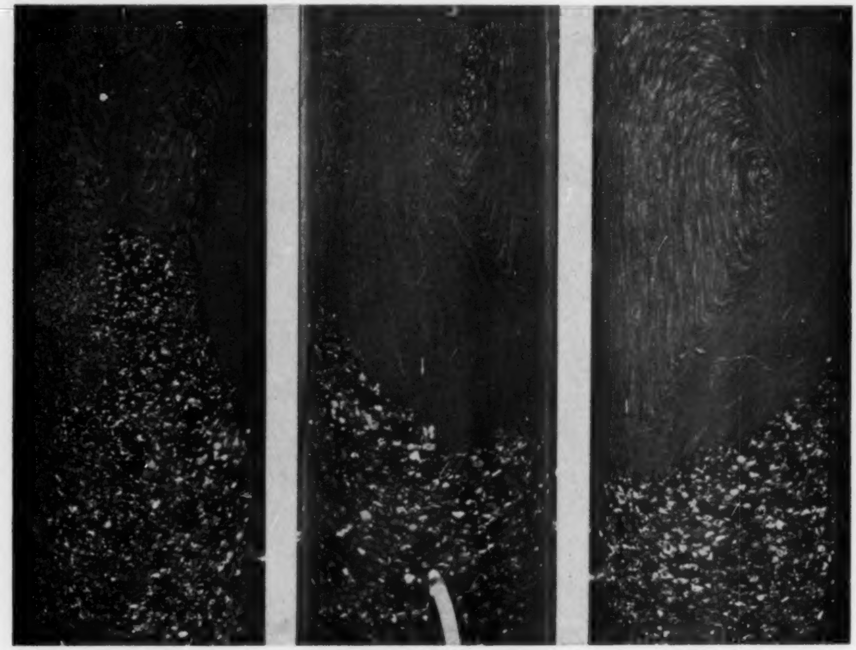

Fig. 7. Uneven Gravel Surface

Note the high mounding of the gravel. The filter was here being backwashed at a rate to produce a 4-in. bed expansion.

gpm/sq ft. The surface of the gravel was definitely beneath its original level, although there was not very much jet action at the gravel surface around the filter wall. Most of the rapid motion of the sand took place about 12–24 in. above the gravel surface. This was most likely produced by an interior sand jet.

A backwash rate of 23.1 gpm/sq ft was required to expand the bed 8 in. Jet action extended around much of the periphery at the gravel surface. There were places where the gravel level was at least an inch below the original surface. Evidently, part of the gravel was being expanded and was in motion with the sand. No sand

Backwashing at a rate of 33.7 gpm/sq ft produced a 12-in. bed expansion. The gravel layer at one place was fully 2 in. below the original surface. A sand jet extended upward about 14 in.

A backwash filtration rate of 36.25 gpm/sq ft was required to produce a 14-in. bed expansion. The gravel surface, shown in Fig. 9, was similar to that at a 12-in. expansion. There was one sand jet that extended about 24 in. upward from the gravel surface. Some streaking occurs in the other photographs of the backwashing at this rate, indicating rapid movement of the sand. No well defined sand jets occurred, except at one place, as mentioned. About as much gravel appeared above the original gravel surface as below it. The filter backwash was then cut off and the bed was allowed to settle. There was not much unevenness of the gravel, but the sand bed was more than an inch above its original level.

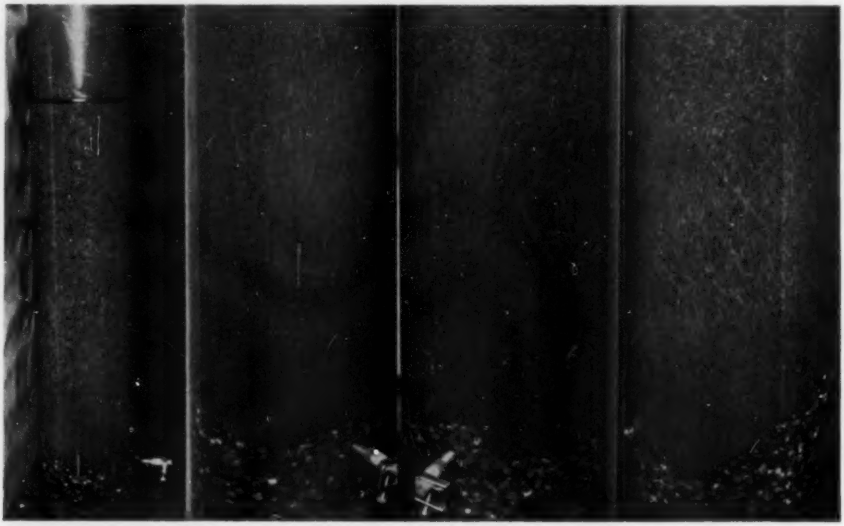

Fig. 8. Backwashing at a 12-in. Bed Expansion

The level of the stationary gravel remained close to the original level, although much of the finer gravel that was caught in the jets finally settled down to produce a slight expansion of the gravel bed.

Three Thin Gravel Layers

After completion of the experiments with two thin layers of gravel, another filter bed was constructed in which the three top layers were each an inch thick. Enough large gravel was added so that the top of the smaller gravel extended to the small takeoff pipe.

Backwashing at a rate of 10.3 gpm/sq ft gave a 2-in. sand expansion. There was almost no jet action at the junction of the sand and gravel, unless it took place away from the side wall of the filter and therefore went undetected.

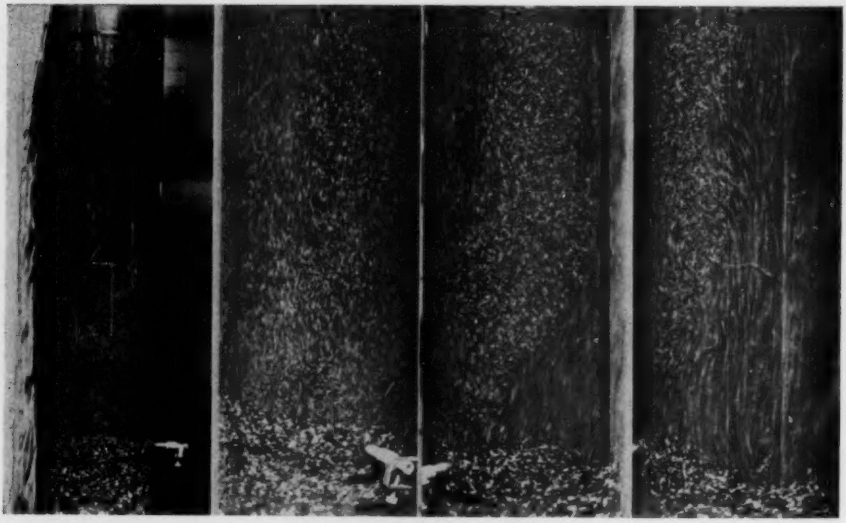

Fig. 9. Bed Expansion of 14 in.

A backwash rate of 36.25 gpm/sq ft was required to produce this bed expansion. Only one large sand jet was clearly visible, as may be seen from the figure.

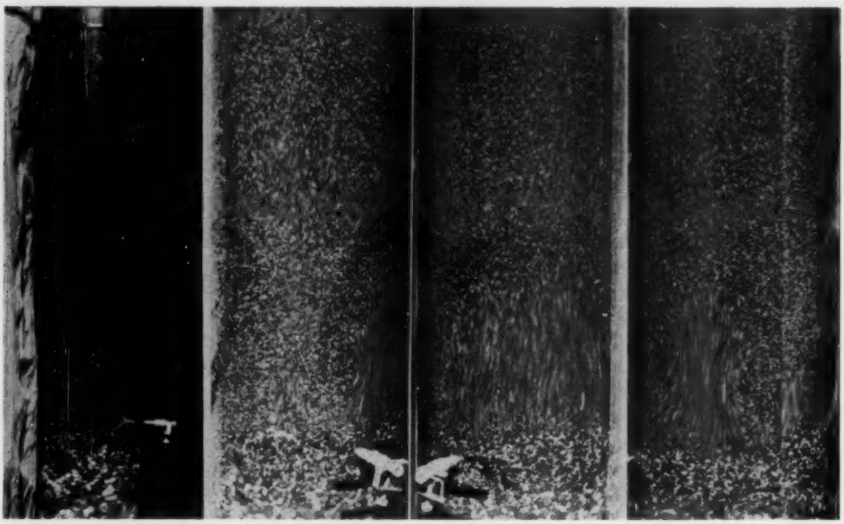

Fig. 10. Bed Expansion of 6 in.

A backwash rate of 17.8 gpm/sq ft was required to produce this bed expansion. The longest jet produced was about 6 in.

Backwashing at a rate of 14.0 gpm/sq ft gave a 4-in. sand expansion. There was some jet action at the junction of the sand and gravel, the longest jet extending upward about 7 in. In addition, there were places in the bed that showed more rapid movement of the sand at higher levels.

The photographs of Fig. 10 show the bed being backwashed at a rate of 17.8 gpm/sq ft to give a 6-in. expansion of the sand bed. In order to make the photographs larger, only that portion of the filter bed in which most of the jet action took place is shown. The longest of the jets shown in the four photographs was about 6 in. (Fig. 10a). There is an area above this jet in which there was little motion of the sand, and another where motion was rapid enough to show streaking. Higher rates of flow were noticed in several places on the other photographs, particularly the upper part of Fig. 10d. The slight, light streaks along the left portion of Fig. 10a and along the right portion of Fig. 10d were caused by reflection from a paper-covered support for the tube.

Backwashing at a rate of 21.25 gpm/sq ft gave 8 in. of expansion. The sand jets did not extend as high as for the 6-in. expansion, with the possible exception of one. There were very few places in the upper part of the filter where there was rapid flow of the sand and water.

Backwashing at a rate of 25.9 gpm/sq ft gave 10 in. of expansion. The maximum height of any sand jet was about 8 in. Near the upper part of the filter was an area showing fairly rapid motion. Particles of gravel mixed with the sand could be detected above the sand jets.

The bed was backwashed at a rate of 30 gpm/sq ft to give a 12-in. expansion. The photographs in Fig. 11 do not show jets extending upward more than 6–8 in. These photographs seem to show the rapid flows of the jets more clearly. They also show that jet origin is in the gravel. This is particularly noticeable in Fig. 11a, b. In Fig. 11c a few particles of gravel are noticed just above the jets, where the motion of the material was not rapid. What appear to be particles of material in some of the jets are believed to be objects fastened to the side of the glass that were not in motion. A particle of gravel in a jet very likely would show as a streak ¼–½ in. long.

Backwashing at a rate of 35.6 gpm/sq ft gave a 15-in. sand expansion. The sand jet action extended upward more than 12 in. Some lowering of the gravel surface was noticed in places where there was greater jet action.

Thin layers of gravel will form the fewer number of mounds, and most likely mounds of lesser height. One reason is that there is less fine gravel to be moved about. In the experimental tube, gravel of ½–¾ in. was only 3 in. from the sand, whereas the filter beds in the South District Filtration Plant have 9½ in. of material less than ½ in. in diameter.

The experiments on the thin-layer gravel beds were backwashing experiments only. Sand leakage would most likely be increased with such thin gravel layers, and it would not be advisable to use very thin layers in a large filter. It is known from plant experience that the finer layers of gravel travel about in the filter bed when they are of the usual thickness, and the tendency in filter bed design, it would seem, should be towards thinner gravel layers, at least for the two top layers.

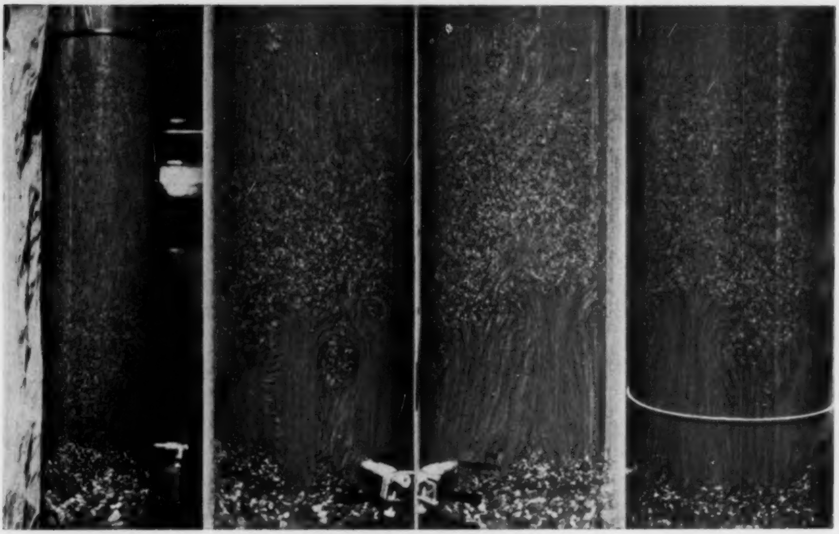

Fig. 11. Bed Expansion of 12 in.

A backwash rate of 30 gpm/sq ft was required to produce this bed expansion. These photographs clearly show the origin of the jet action to be in the gravel.

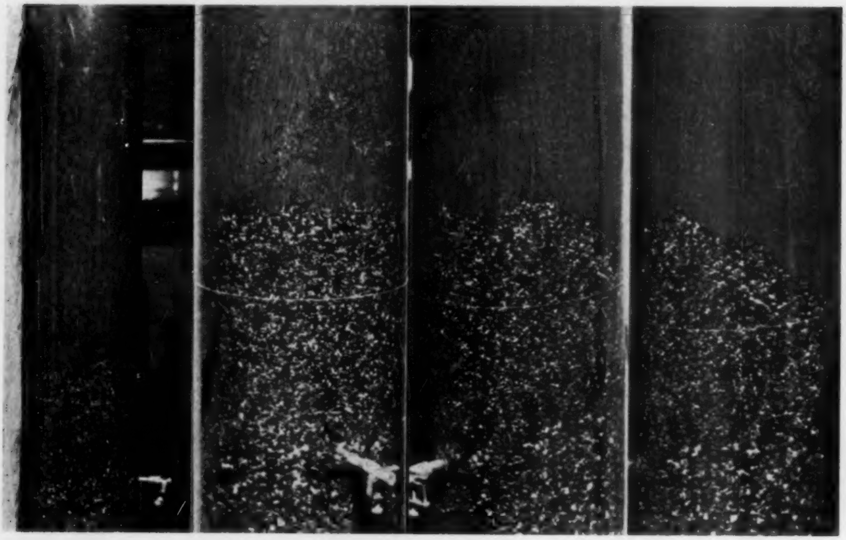

Fig. 12. Bed Expansion of 10 in.

The string represents the original level of the gravel before backwashing. The average level after backwashing at this rate was about 2 in. above the string.

Thick Layers of Fine Gravel

The experiments with thin gravel layers suggested the need for a few tests with thick layers of fine gravel. There was evidence of expansion of the fine gravel layers after backwashing a filter, and one purpose of building a filter with a thick layer of fine gravel was to determine the amount of expansion after backwashing. A string tied around the filter represented the original top of the gravel.

At a 2-in. bed expansion, gravel appeared at places about an inch above the line at the top of gravel, 2 in. being the maximum. Sand jet action was limited to several small jets.

At a 4-in. bed expansion, the gravel not in motion was at a lesser height than at a 2-in. bed expansion. Sand jets were more numerous and at one place extended upwards about 10 in.

At a 6-in. bed expansion, the gravel bed at the level where jet action occurred was only about $\frac{1}{2}$ in. above the original height, and was about the same all around the filter. Around more than half of the filter, the sand jets extended upwards for 4–6 in.; then there was a layer of a mixture of sand and gravel about 2 in. high. It appeared to be mostly sand that was nearly still. Above this layer was another ring of jets extending upward 6–10 in. This most likely would not have occurred in a large filter bed.

At an 8-in. expansion the gravel surface of the filter averaged about $\frac{3}{4}$ in. above the original level, with more variation in level. Some of the sand jets extended upward more than 12 in.

The backwashing rate was increased to produce a 10-in. expansion. The gravel expansion became greater, and the top of the gravel more uneven. Figure 12 shows the condition at a 10-in. bed expansion. The gravel surface varied from the string (representing the original level) to slightly more than 2 in. above the string. The average level was more than an inch above the original.

At a 12-in. expansion, the level of the top gravel was similar to what it was at lower backwash rates, and extended from the string to about 3 in. above. The average level was about 2 in. above the string. Level variations of the gravel layer of only 6-in. diameter were considerable. The gravel would probably pile to greater heights in a large filter. The sand jets extended upward over 12 in. for the longer jets.

Large Gravel Expansion

The tests with a 6-in. layer of $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel gave strong evidence of expansion in the finer gravel layers. The fact that at a 12-in. bed expansion about 50 per cent of the gravel around the periphery averaged 2 in. above the original level means that the average height over the entire bed must have averaged more than an inch, and probably at least $1\frac{1}{2}$ in. Nearly all of this expansion occurred in the 6-in. layer of $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel. If the voids in the gravel of this size were 50 per cent, they occupied space equivalent to 3 in. depth of the 6-in. layer. To increase the voids by an inch of additional depth would increase the void area by one-third. These increases may be quite large when in the form of larger channels through the gravel.

A bed of $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel started to expand at about a 35-gpm/sq ft backwash rate, and the gravel almost expanded just as would sand. At places where there was jet action in the sand, gravel was picked up rapidly in the jets, carried a few inches upward with the sand and water, and deposited at other places in the bed.

The amounts of water and sand in any jet of more than 2–3 in. height are great enough at low backwash rates to transport top-layer gravel to other places in the bed. The higher backwash rate merely increases the rate of transfer. A backwash rate that produces real suspension of the gravel in the top layer tends to redistribute the gravel over the surface. This, however, does not prevent gravel mounding, for the origin of jets is always at the surface of the material not in motion.

Two $4\frac{1}{2}$-in. Layers of Gravel

The next experiment with the 6-in. plastic tube filter was to construct a bed with a top $4\frac{1}{2}$-in. layer of $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel, and $4\frac{1}{2}$ in. of $\frac{1}{8}$- to $\frac{1}{4}$-in. gravel. At a bed expansion of 2 in. there were no sand jets around the wall of the filter.

With a bed expansion of 4 in. there were a few jets extending upward several inches. The gravel level remained at almost the same level as when constructed.

When backwashed to give a bed expansion of 6 in., the gravel remained at about the original level. Some sand jets originated at the gravel surface, and others higher up.

At a bed expansion of 8 in., the sand jets originating at the gravel surface were not very numerous and did not extend very far upward. Jets originating higher up in the bed were more numerous and of greater height. The gravel still had not expanded an appreciable amount.

When backwashed to give a 10-in. bed expansion, the gravel expanded for the first time, and the expansion was not very great. The sand jets originating at the gravel surface were more numerous and extended to a greater height—about 10–12 in. for some of the jets.

At a 12-in. bed expansion the gravel expanded a little more than for 10 in. The sand jets rapidly extended 6–8 in. upward.

After backwashing at a 14-in. bed expansion, there was more mounding of the gravel surface than at lower rates. There still was not much expansion of the gravel. It may be that only the $\frac{1}{12}$- to $\frac{1}{8}$-in. gravel expanded, and with only $4\frac{1}{2}$ in. of this size the gravel expansion should be less than for a 6-in. layer. It is possible that mounding did take place away from the filter wall and could not be observed.

The backwashing tests with two $4\frac{1}{2}$-in. layers of gravel did not show as much expansion of the gravel bed as expected. There was one difference in the bed from the ones used in previous tests. The gravel was rescreened by hand sieves ($\frac{1}{12}$ in.). A considerable amount of the gravel passed through the sieve. The $\frac{1}{8}$- to $\frac{1}{4}$-in. size also was rescreened through the $\frac{1}{8}$-inch sieve. The sand was taken from a new carload shipment recently received. All of this may have caused the difference from previous tests in the expansion of the gravel.

Additional Backwashing

One might conclude from the tests that two layers of gravel, each $4\frac{1}{2}$ in. thick for the top sizes of the gravel in a filter, is the better gravel grading to use. Had the backwashing experiments been stopped at this series of tests, such a conclusion would be justified. A little more backwashing, however, at a 14-in. expansion, produced

considerable permanent expansion of the gravel. There was considerable expansion of the gravel over the original level—about 4 in. at the highest point. Particles of gravel were thrown up into the sand by jet action.

The backwash was cut off, and measurements indicated not more than $\frac{1}{4}$-in. settlement of the gravel, and most of the measurements show only about $\frac{1}{8}$-in. settlement. The number of visible grains of sand scattered through the gravel was almost alarming. The bed was designed to be as free as possible of sand, but the sand seemed gradually to work its way down through the gravel even with no filtration. Slow leakage of sand through filter beds is more widespread than generally believed. Such leakage is distinguished from rapid leakage in that slow leakage forms no holes in the sand. In a filter with approximately 1,400 sq ft of filter surface, the loss may be about half of a cup, more or less, for each filter run. This means so little loss that it may well be that more is washed over the washwater troughs than through leakage.

Uneven Gravel Level

The 6-in. plastic tube filter was again examined. It had stood idle about one week. The height of the gravel above the cord marking its original level was 6-in. on the southwest side, and $2\frac{1}{2}$ in. on the north side. The sand was at an elevation $1\frac{1}{2}$ in. above its original level. If the gravel within the bed was the same level as that around the wall of the filter, the average permanent gravel expansion was 4 in. This is evidence that part of the coarser material in the sand was mixed with the gravel. Even with no mixing, the sand bed surface should have been more than $1\frac{1}{2}$ in. higher than when constructed. The photographs, however, do not indicate much mixture of the coarse sand with the top layer of gravel.

The encouraging results when the filter was first washed to expand the bed to 14 in. did not last. With a $3\frac{1}{2}$-in. variation in gravel height in a 6-in. diameter bed, much greater differences can be expected in larger beds.

Glass Side Filter

A small, steel tank filter with one glass side was constructed for use at the experimental filtration plant in 1938. The filter is 9 sq ft in area and $8\frac{1}{2}$ ft deep. This unit was transferred to the South District Filtration Plant when the experimental work was finished. It has been used in experimental work at the South District plant.

When the experiments on backwashing were started, it was only natural that this filter be used, since it is considerably larger in area than the 6-in. plastic pipe filter. Its main disadvantage is that only one side of the filter is glass, and it is necessary to guess at what may be taking place on the other three sides. Another limitation is that the photographs are not clear.

Heavy Gravel

Before the backwashing tests were started, this filter had been constructed with $\frac{1}{12}$–$\frac{1}{8}$-in. and $\frac{1}{8}$–$\frac{1}{4}$-in. gravel which was replaced with a crushed iron ore of 3.30 sp gr. The angular shape of the particles probably nullified some of the benefits of a heavier material. The iron ore had been in the unit several months before these tests were

started. Except for the substitution of crushed iron ore for gravel in the two top layers, the filter was the same as the large filters, as far as the size and depth of coarse aggregate layers and the depth of sand are concerned. The backwash was run continuously for 24 hr at each backwash rate.

Backwash Rates

When the filter was backwashed at a rate to expand the bed 4 in., the difference in coarse aggregate level along the glass side was 2 in. The amount of backwashing after the bed was constructed was not known exactly. The filter had been used in regular filtration experiments for several months. The variation in coarse aggregate level increased gradually.

Fig. 13. Sand Boil at Filter Surface

This photograph (flashbulb) was taken while the filter was being backwashed at a 2-in. sand expansion. The boil extended 3–4 in. above the expanded bed.

There was almost a continuous line of jets along the glass wall; they extended upwards about 6 in., with one jet extending almost to the sand surface.

At a 6-in. sand expansion, the line of sand jets along the glass side was continuous. Some of the jets were nearly 12 in. long. Some rapid motion of the sand took place near the top of the bed. The aggregate had built up slightly higher on the left side, and was slightly lower on the right side, than at 4-in. expansion.

The filter was backwashed to give an 8-in. bed expansion, and the top of the aggregate indicated movement from the back toward the front. Sand jet action occurred over much of the glass side, with some of the jets extending almost to the top of the bed.

When the filter was backwashed at a rate to give a 9-in. expansion, there was a difference in elevation of 7 in. within a horizontal distance of about $1\frac{1}{2}$ ft. Most of the side of the bed that was visible through the glass was streaked or slightly streaked in pictures taken at a $\frac{1}{2}$-sec exposure. This indicates that the velocity of the water in the jets was quite high.

With the backwash rate sufficient to give a 12-in. bed expansion, the height of the coarse aggregate diminished on the left and stayed about the same on the right. The difference in elevation over the top of the aggregate was about 6 in.

When the filter was backwashed to give 15-in. expansion, there was not much change in the aggregate level. The total variation was about 4 in.

The use of material of somewhat greater specific gravity than gravel is apparently not the solution to the problem of gravel movement in the filter bed. It is of some value, but gravel still can be moved about by sand jet action. The specific gravity of the aggregate would probably have to be nearly twice that of gravel to avoid motion. There is no possibility of being able to acquire such a high-specific-gravity material at a price rea-

sonable enough for use in large filter beds.

Filter 81 Reconstructed

Between Nov. 1 and 17, 1957, the bed in Filter 81 was reconstructed. The high-specific-gravity material was removed from the test filter and it was made up with gravel and sand of the same sizes and depths as the filters in the South District Filtration Plant.

At a sand expansion of 2 in., there was only one place where a sand jet

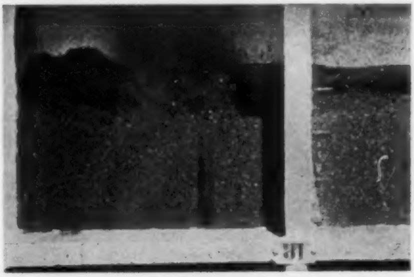

Fig. 14. Sand Being Kicked up Into the Water

The boil at the left rear of the filter was not as large as at a 2-in. expansion (this picture was taken at an 8-in. expansion). A flashbulb was used to catch individual sand grains.

was noticed. The height of the jet was about 6 in. Gravel was being moved upwards at one place, and there was a sand boil in the far back corner of the filter. This is shown in Fig. 13. The filter was being backwashed to give a 2-in. bed expansion. The boil extended 3–4 in. above the top of the expanded bed.

Figure 14 is a photograph of the filter being backwashed to produce an 8-in. bed expansion. The boil in the far back corner was not as high as it had been at a 2-in. bed expansion. One characteristic of boils is that sand is kicked up into the water a few inches above the surface of the expanded bed. This phenomenon was observed at length and always occurred at the edge of a sand jet that extended from the gravel to the surface of the expanded bed. Such sand jets are commonly called "sand boils." The photographs were made with flashbulb exposures so as to catch individual sand grains. *This kicking of loose sand grains up above the expanded sand surface is the cause of the loss of much sand in many filtration plants.* Not all sand boils will kick sand from the bed into the water above. Why the sand leaves the boil at some point around the edge of the boil is difficult to understand. At such places water thrusts upward with sufficient force to take with it grains of sand. The loose grains of sand are detected near the plate glass to the right of the boil on the left hand side of the filter.

The upper photograph in Fig. 15 was taken at a backwash rate sufficient to expand the bed 6 in. The photograph shows the pointed origins of the sand jets at the surface of the gravel. Note the ridges and valleys in the gravel surface. The variation in the level of the gravel surface was 3 in.

The lower photograph of Fig. 15 was taken with a backwash rate that expanded the bed 8 in. Many filtration plants backwash filters so as to give 25–30 per cent expansion of the sand beds. The conditions occurring at 6–8 in. expansion of the sand bed are therefore typical of what occurs in actual practice. In a new, accurately constructed bed, backwashing starts to move the top gravel around the first time it is backwashed. In Fig.

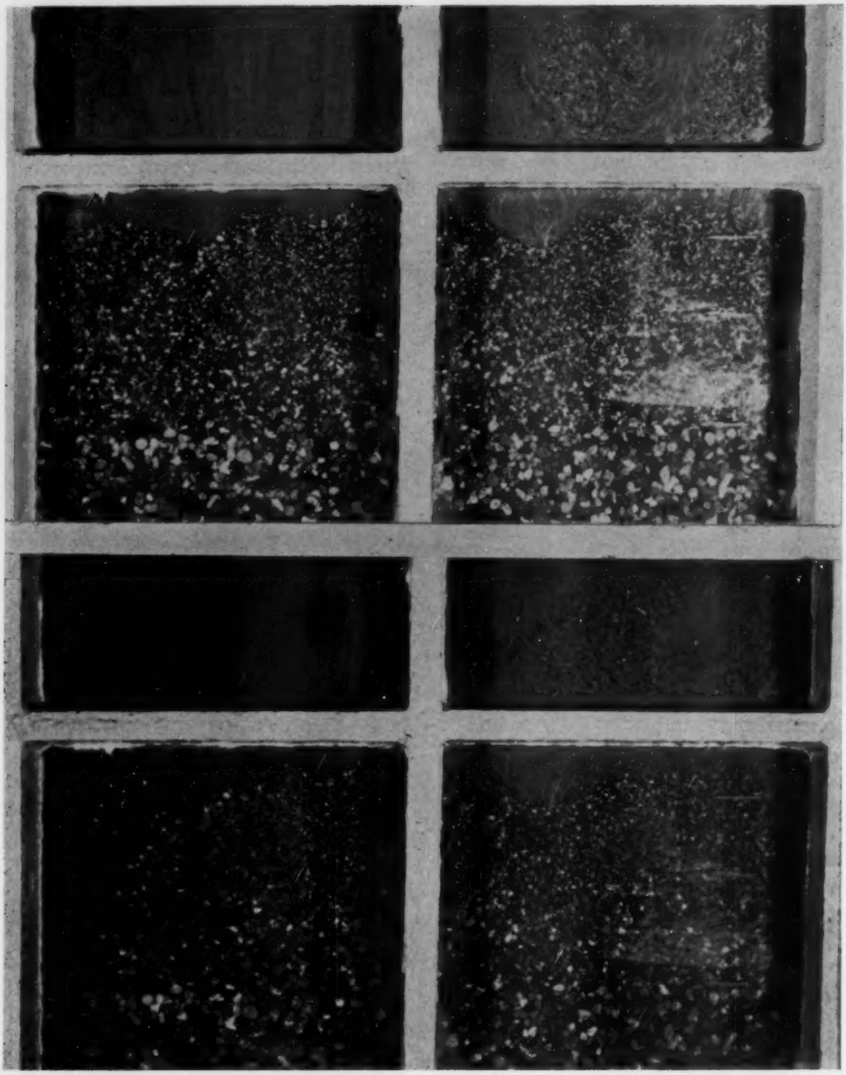

Fig. 15. Backwashing of Glass Side Filter at 6- and 8-in. Bed Expansions

Upper photograph, 6-in. expansion; lower photograph, 8-in. expansion. Here the ridging of the gravel is more clearly shown than on other photographs.

15, the ridging produced by the sand jets is more plainly visible than in some of the other photographs. The time of exposure in these photographs was ⅕ sec.

At a 10-in. expansion of the bed, the sand jets extend along almost all the sides of the filter. The highest jet extends upward about 14 in. Also there are places in the upper part of the bed that indicate rapid motion of the sand.

When the filter was backwashed at a rate to give 12-in. expansion of the bed, the sand jet action extends across the side of the filter bed, and up to a height of nearly two feet for the highest. In general, the gravel surface is above the original level. There is some indication that finer gravel is concentrated in parts of the gravel bed, and in other parts, particularly under the higher sand jets, the gravel seems to be of larger size. This would make the upward passage of water easier. Therein may lie the major cause of such large jets.

A 14-in. sand expansion was the maximum that could be produced with the backwash piping system. There were sand jets along the entire glass side, some of which extended upward more than 24 in. The backwash rate may have been great enough to suspend some of the gravel. The areas of fine gravel concentration seem to be cleared of some of the fine gravel noticeable at 12 in. expansion.

Specific Gravity of Sand and Water Mixture

To keep sand particles suspended in water, as takes place in backwashing of filters, the sand and water act as a fluid with specific gravity greater than unity. This can be observed by inserting a tube in the filter bed all the way to the bottom of the sand. If there is 24 in. of sand in the bed before the backwash is started, the water in the tube will rise above the surrounding water, which shows that the mixture of sand and water is heavier than water alone.

It is assumed that the sand has sp gr 2.60 (the specific gravity of silica sand varies from 2.60 to 2.65), and that voids in the sand take up 50 per cent of the volume. Generally the voids are close to 45 per cent. A cubic foot of solid sand without voids at sp gr 2.60 will weigh 162.5 lb. If in the form of grains, sand is 50 per cent solid, then its weight is 81.25 lb/cu ft. The weight of 0.5 cu ft water is 31.25 lb. The sum of the weight of gravel and water equals 112.5 lb/cu ft. This is equivalent to sp gr 1.80 for mixture of water and sand. If the percentage of solid sand is 25, and the percentage of water is 75, the specific gravity is theoretically 1.40. The fact that the mixture is in motion, and probably cannot be in uniform motion, explains why its specific gravity may vary from its theoretical value.

After a 6-in. expansion of the bed, a 24-in. bed then would be 30 in. deep. The mixture would be 40 per cent solid sand and 60 per cent water. It is easy to determine that the specific gravity of the expanded sand is 1.64.

A method has not yet been found to test the ratio of sand to water in the jets at the boundary between the sand and gravel. It would be unlikely for the sand to be more than 25 per cent, and some of the jets may be only 10–15 per cent solid sand.

The water flows upward through the gravel where the jet is taking place, and enters the sand at a high velocity which has been measured to be in excess of a foot per second for some of the jets. It produces a mass within the jet in the expanded sand bed that had a lower specific gravity than the surrounding sand and water. Now if it can be assumed that the weight of the sand and water within the jet is considerably less than the weight of the expanded sand in the bed, the hydraulic conditions produce sufficient head differential to cause the high jet velocity.

TABLE 2

Loss of Head Owing to Friction

Backwash Rate *gpm/sq ft*	Head Loss—*ft*			
	$\frac{1}{12}$–$\frac{1}{8}$-in. Gravel	$\frac{1}{8}$–$\frac{1}{4}$-in. Gravel	$\frac{1}{4}$–$\frac{1}{2}$-in. Gravel	$\frac{1}{2}$–$\frac{3}{4}$-in. Gravel
20	1.34	0.55	0.25	0.10
30	2.80	1.00	0.47	0.15

Flow Through Gravel

For beds of gravel 24 in. deep that were backwashed at various rates, the friction loss is as shown in Table 2. It is evident from the table that it is not difficult for water to flow through the gravel rapidly enough to supply all the water needed for a sand jet. Experimental evidence indicates that the $\frac{1}{12}$–$\frac{1}{8}$-in. and $\frac{1}{8}$–$\frac{1}{4}$-in. gravels soon become mixed and together act as a gravel of larger size than the $\frac{1}{12}$–$\frac{1}{8}$-in. gravel. Much of it is so rearranged in a filter bed that the percentage of voids is increased, as is evident from the expansions found in experiments in glass or plastic side filters. Sand mixed with the gravel would increase the head loss as shown in Fig. 3.

Principles of Sand Jet Action

If all gravel particles were of the same diameter, true spheres, and if they were placed in a box in regular order with one particle directly over another, the voids would amount to 47.5 per cent of the total volume. Arranged in the most compact way possible, the percentage of voids would be somewhat less than this. Gravel particles are not true spheres, and there is much variation in the diameter for a given layer in a filter bed. Generally, filter bed gravel is considered to be about 50 per cent void. The percentage may be higher if the gravel is very irregular.

For purposes of this discussion a void area of 50 per cent is assumed. Any disturbance in the gravel, such as an excessively high backwash rate, turning on the backwash too rapidly, or other causes of slight movements of the gravel, will keep the gravel from settling back compactly. Instead of a void area of 50 per cent, it may be 60 or higher in the places where the gravel has been disturbed, making it still easier for localized, rapid flows to take place.

From photographs of the top gravel, one would guess that the void area was in excess of 60 per cent. In one filter bed, backwashed for the first time at a rate that produced some movement of the gravel, the $\frac{1}{12}$–$\frac{1}{8}$-in. gravel expanded $1\frac{1}{2}$ in. in a layer 18 in. thick, or 8.33 per cent over the initial volume. The voids were 50 per cent when the filter was placed in service, and after the backwashing they were

about 58 per cent. After a prolonged backwashing of the filter, some observations indicated a 10–15 per cent or greater increase in the gravel volume.

All of this may not represent increase in void area, for some sand may be mixed with the gravel. The observations made with glass or plastic filter tubes do not indicate that sand is mixed with the gravel, but in the large filter beds there is mixing in places.

A bubble of air, turned loose beneath the surface of water, quickly makes its way to the surface. A mixture of water and sand with sp gr 1.2, turned loose at the bottom of the sand bed in a filter being backwashed with expansion to give sp gr 1.6 to the mass, also will move upwards at a velocity considerably in excess of the upward velocity of the water through the sand at other places in the bed. In other words, the tendency of lighter masses to rise to the surface of a fluid of heavier mass is the cause of jets.

The sand jet has its lowest specific gravity and highest velocity just as it leaves the gravel not in motion. Because no sand has yet been mixed with the water, the specific gravity should be 1.0 at this time. It strikes the sand and particles of small gravel being pushed into the area of the sand jet, and they are carried upward with the jet. The sand jet then strikes the more compact sand and water above, and its velocity begins to be reduced. Small jets with low velocity may extend upward only 3–4 in. This is true with only a 2–4-in. expansion of the sand bed. As expansion is increased by higher backwash rates, the jets leave the nonmoving gravel at higher rates and extend a greater distance into the expanded sand, perhaps 6–12 in. for sand bed expansions of 6–8 in. Some of the sand jets will extend to a height less than 6 in., and some may extend to a height greater than 12 in. Greater expansion of the sand causes it to offer less resistance to the upward progress of the jet.

At some initial jet velocity, particles of gravel are moved, and these particles speed upward until the velocity of the sand jet is checked. Then they begin to fall. Many of the gravel particles fall back into the jet and are again carried upward. Gravel particles may be carried upward time and time again, but some of them are eventually thrown so far away they remain wherever they fall, forming a gravel mound. Often this process builds up mound deposits composed of both gravel and sand that offer resistance to the upward flow of water. This may allow some of the water in the expanded material above to flow out, thereby increasing the specific gravity of the material that is in slow motion.

The movement of gravel in a filter bed by jet action is not a rapid process. When the filter is backwashed only a few particles of gravel are moved from the jet region. The next time the filter is backwashed a few more are moved, and so on. As soon as a gravel particle moves out of the influence of a jet it begins to descend, settling back on the motionless gravel, or sand and gravel. Sand jets must be supplied with a constant flow of sand. The sand, with a few particles of gravel, may recirculate three or four times during the 3–4-min backwash period. A small percentage of the gravel is thrown far enough from the place of jet action not to be drawn back and recirculated.

Mounds of mixed sand and gravel offer more resistance to the flow of water than sand alone, and tend to divert the water to places where jets are flowing, thereby increasing the size of the jets, and often their velocity. More and larger stones may then be lifted by the jet. In large filters, the $\frac{1}{8}$-in. gravel particles are moved first; then, when this size has been removed from the jet area, the jet force usually becomes strong enough to lift $\frac{1}{8}$–$\frac{1}{4}$-in. particles. The time required for going through this layer may be several years.

With the two finer sizes of gravel removed, the resistance to flow through the lower and larger gravel is not so great. This also tends to move more and larger gravel particles from the crater, and causes some of the craters to extend into and occasionally through the third gravel layer.

Voids in the $\frac{1}{4}$–$\frac{1}{2}$-in. gravel are large enough to allow leakage of sand. At first it usually is slow leakage, and no holes develop in the sand bed. The gravel may, however, become so rearranged by the rapid jet flow that a stream of sand passes downward through the gravel, forming a hole from the sand surface to the gravel. This is called rapid leakage of sand.

The author did not conduct tests with anthracite coal. It should be evident that coal would be less likely to move gravel particles, though it might not prevent its movement entirely. Crushed coal is used in a number of filters throughout the United States, and its use may increase. It has some disadvantages as a filtering material that will probably prevent its universal use. Because of its gradual loss, it will no doubt prove to be more expensive than sand, even though sand beds have to be rebuilt occasionally. Sand grains never wear smaller, whereas coal slowly decreases in size if surface washing is used.

Sand Leakage

In 1953, 6 years after they had been placed in service, several of the filters in the South District Filtration Plant were found to have greater sand loss than expected. It was known that some of the sand had been lost over the wash water troughs during backwashing, but there were also indications of sand leakage through some of the beds.

Sand leakage can be detected at the South District plant, because sand will be deposited in the filtered-water basin in a manner that shows the filter from which it came. Although this determination can be made by dewatering a basin, such action requires cutting off 20 filters, which can be done only in two or three of the winter months, when the filter runs are longer than average.

Filtered-water basin No. 1, serving Filters 1–20, was drained on Dec. 14, 1953, and inspection of the basin revealed that some sand had passed through three of the filters. The amount varied from about a cubic foot to 3–4 cu yd of sand for the worst filter. Except for Filters 10 and 11, the loss of sand was not enough to worry about, as far as cost is concerned. The passage of sand through a filter bed causes more concern because it may start to get worse as the years go by. By 1953 all of the sand beds were examined to see if evidence of sand leakage could be found. If leakage is bad, holes form through the bed and extend to the same surface; the best time to watch for such conditions is just before washing a filter.

In the notes on filter operation for Jul. 27, 1953, reference is made to an examination of the sand surface of Filter 71. The notes state that one sunken place in the sand was found at the back of the filter. This later was found to be caused by turning on the filter influent too rapidly after washing the filter and was not of real significance. It did cause more frequent examination of the filter bed. A hole through the sand was found 2 days later, indicating sand leakage. The hole represented the loss of about a pint of sand. The sand and gravel were removed for a few feet around the hole.

Close to the hole there was a mound of gravel which extended upward about 12 in. above the original gravel level. This mound was almost like a chimney, 6–12 in. or more in diameter. At the time it was thought the sand leakage was passing through the mound of gravel. Later studies have indicated that the mound is built up by deposits of gravel washed from a place where there is sand jet action. Jets that extend to the surface of the expanded sand bed and form boils are most likely to produce gravel mounds. The particles of gravel are moved up into the expanded bed from the gravel surface where the high jets originate, and some are deposited along the side of the boil. Examination of the bottom of this filter failed to reveal a broken lateral, or an enlarged hole that might cause an unusual boil. The filtering material was replaced and the filter was put back in service.

Two Layers of Heavy Material

Additional work was performed on Filter 71 in 1955. The sand and the two top layers of gravel were removed hydraulically and two layers of a crushed black material produced from the dust of flue gases of a high-pressure boiler plant were added. This material has a specific gravity of about 3, and has thus far left the surface at the bottom of the sand in a level condition. It is hoped that the heavier material will not mound.

When Filter 41 was probed with a stick during backwashing the fine gravel was found to have moved largely to the east side of the bed. The surface of the gravel as constructed was at elevation plus 5.3 ft. The elevations of the gravel at different parts of the bed varied. Elevations over plus 5.3 ft represent mounding, and figures under 5.3 represent places where some of the gravel had been moved. The gravel was highest along the east wall of the filter. Elevation 6.0 ft, found in several places, is 0.7 ft above the constructed level.

When a sand boil near a filter sidewall moves gravel, the gravel which falls by the side of the wall is less likely to be disturbed in backwashing. Mounds that form away from the sidewall are more likely to be washed away as time passes, particularly in the early stages of formation. Only a few mounds survive and build up to large size. For most of the area of the bed of Filter 41, the gravel levels were lower, which may have been because the probing stick extended below the true gravel surface.

A memorandum of the work on Filter 41 states that sand was removed from the filter by the laborers by moving westward until gravel first occurred at the plus 5.3 elevation. Pulling back of sand was stopped and the exposed gravel along the east side of the filter was removed to the 5.3 elevation. The 24½ in. of sand was relaid

over the gravel. About 15 cu yd of gravel (mixed with 30–50 per cent sand) was removed from the filter. From the probing elevations obtained there was apparently an easterly shift of the gravel, leaving the west side of the filter with a large layer of sand and a deficient gravel elevation.

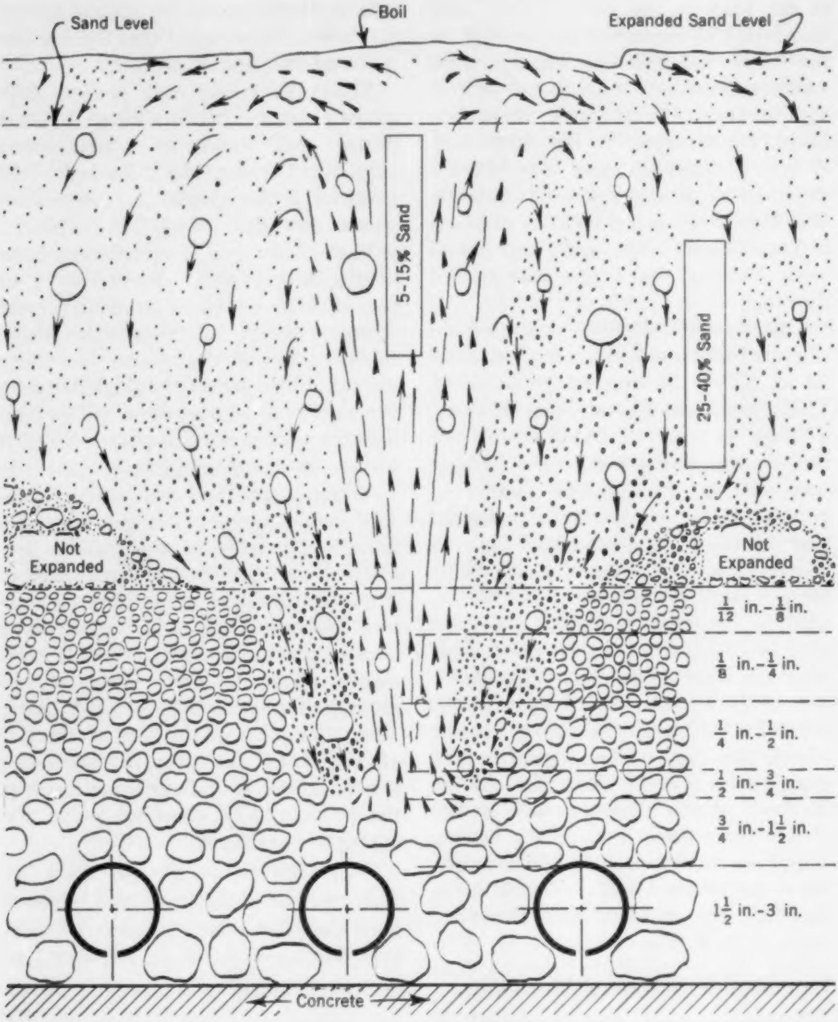

Fig. 16. Sand Boil

The drawing is based upon data gathered from the photographs in the experiments. Note the downward motion of the sand directly to the side of the boil.

Removal of the mounded gravel required about 49 man-days, and cost about $700. The removed gravel was later sieved and the material stored for future use. None of it was returned to the filter. The filter has functioned satisfactorily since removal of the mounded gravel. In February, 1954, a memorandum on the gravel levels of the 10 high-rate filters stated that Filter 41, which was rebuilt in 1953, was the most consistent of the filters and had the least variation in gravel elevation. The high cost of removing a relatively small amount of gravel from a filter and moving the sand over about half of the bed area made it desirable to install permanent equipment for handling sand and gravel.

Filter Bed Conditions in 1947

Up to the end of 1947 no filter bed in the plant had been rebuilt, unless Filter 71 could be considered a rebuilt bed. Until 1955 there had been no damage requiring that this bed be rebuilt. The reason for rebuilding was to try a heavier material for the top gravel layers.

Filter 31 developed the first sand leakage hole of any consequence in September 1957. More than a cubic foot of sand passed down through the gravel between backwashes. The filter was being drained for backwashing when the hole was noticed along the back wall near the center. The bed was not rebuilt because of this hole, although the sand and gravel were removed for a few feet surrounding it. Again, no defect could be found in the underdrains, and the only conclusion was that the gravel was being moved by jet action to a depth that allowed sand to pass through the gravel. There were several pieces of gravel about an inch in diameter at the bottom of the crater.

On Feb. 27, 1958, clear-water basin No. 3 was drained. It was inspected the following day. A small amount of sand had passed through Filters 41 and 51, probably not more than a cubic yard total for the two filters. This is a very good record for filters in use 10.3 yr.

Inspection of the manifolds of Filters 1–5 and 11–15 was made on Nov. 18, 1955, during a period when the backwash line was cut off for insertion of a new backwash valve. Filters 2 and 4 both had about 2 cu ft of sand in the manifold. About 3 cu yd of sand were found in the manifold of Filter 11. Traces of sand were found in the manifolds of Filters 14 and 15. On Dec. 15, 1955, inspection was made of the manifolds of Filters 6–10 and 16–20. About 2 cu yd of sand was found in the manifold of Filter 10.

The manifolds of Filters 21–25 and 31–35 were examined Jan. 31, 1956. A large amount of sand (about 10 cu yd) was found in the manifold of Filter 31.

Filter 71 has shown considerable sand leakage.

More than half of the filters have been inspected for sand leakage. It has been found in eight of the filters. It is guessed that perhaps twelve filters are leaking sand in various amounts. This is not a bad record, and may be what can be expected for filters of good design.

Sand Boils

The author has never seen a filter free of sand boils. Two types of boils may occur in a filter bed. One takes place at the beginning of a backwash in which the wash water rushes into the filter so rapidly that it lifts the bed

of sand and leaves a layer of water with very little sand in it just above the gravel. This layer of water breaks through the bed in places and causes large boils before the bed becomes expanded.

The second type of boil is caused by the formation of jets of water and sand that are of such magnitude and velocity that they extend from the gravel surface to the surface of the expanded bed. Unlike the usual sand jet which is an inch or less in diameter and close to its point of origin, these jets may have a number of points of origin so close together that they join to form a cylinder of water several inches in diameter, with a force large enough to rise to the top of the expanded sand. The boil may be 4–12 in. or more in diameter when it reaches the surface, and it will generally occur in the same place at each backwashing. Figure 16 illustrates a jet-formed sand boil.

A jet is a mixture of sand and water traveling upward at a velocity of 0.3–1.0 fps. The velocity may be much greater than 1.0 fps in some sand jets, particularly in sand boils. Water with no sand passes upward in backwashing through the stationary gravel and picks up sand at the top of the gravel. The partly expanded sand tends to squeeze in on the sand jet from all sides, but the rapid velocity keeps washing away the sand and maintains the volume of the jet at about the same size throughout backwashing. If the sand jet is not of sufficient velocity to lift gravel particles, then the gravel remains in place. This is what should happen, ideally, in filter backwashing.

The head of water required to maintain a sand bed 24 in. deep in partial suspension is approximately 1.8 ft, which gives a theoretical velocity head, not counting friction losses, of about 10 fps. Such a force is capable of doing great damage to a filter bed.

When the filter bed is first placed in operation, the gravel layers are level and no boils of the jet type exist. Sand jet action takes place in the lower part of the sand bed during each backwash period. Some of the jets occur at the same point during each backwash; others travel about over the bed, some very slowly, others more rapidly. Fixed sand jets do most of the damage. A jet that extends upward into the expanded bed more than 3–4 in. generally has a high enough velocity to move fine particles of gravel. A few particles of gravel are moved at each backwash, and a few of these may fall on another part of the gravel surface where they do not come under the influence of the sand jet at the next backwash period. For all sand jets having a fixed point of origin, the next backwash period moves a few more gravel particles, again distributing some of them outside of the influence of the jet.

The sand surrounding the jet moves toward it and is drawn into it. If sand moves upward at the jet locations, it must move downward between them; otherwise the sand soon would be riding on top of a layer of water—which cannot happen once the bed becomes expanded. It is estimated that the sand occupies 10–15 per cent of the volume of the sand and water in a sand jet. Where it is falling, the sand occupies 25–40 per cent of the volume, and probably more than 40 per cent in a few instances. The mass of sand and water at the side of a sand jet therefore has a greater specific gravity than that within the jet, and

this gives the buoyant force that causes the rapid velocity of the jet.

Many sand jets have such a velocity that they readily move gravel particles. Most of the gravel particles recirculate in the bed, returning with the sand to feed the jets. In time—a few weeks, months, or years—a hole is formed that may extend several inches below the original gravel surface. There is evidence that in the building up deposits of gravel and sand a little distance from the location of a sand jet the mixture compacts into a mass that is not expanded by the backwash. Most of the water that should be going upward through this deposited material is deflected to areas of lower resistance. As gravel is washed away the flow within the jet tends to increase, causing it to pick up material of still larger size. Little by little the hole in the gravel becomes deeper, the sand jet larger, and the velocity greater. Eventually the rapid velocity of the jet (probably less than 10 per cent of sand, and a little gravel) forces it to the surface of the expanded bed and spreads it in a typical boiling action. Jets are not always exactly vertical, which may throw the gravel a greater distance from the crater and form larger mounded areas on one side of the boil. Often the material from a sand boil covers up other areas of jet action and makes them inactive. This may deflect more water to the jet or jets producing the sand boil. There is evidence that large sand boils may be fed by a number of close jets acting in unison.

First the $\frac{1}{12}$–$\frac{1}{8}$-in. gravel, if used, is moved. As the jet becomes more forceful, the $\frac{1}{8}$–$\frac{1}{4}$-in. gravel is also moved. Small sand boils usually are forceful enough to move gravel of $\frac{1}{4}$–$\frac{1}{2}$-in. diameter, whereas larger sand boils may move gravel of $\frac{1}{2}$–1-in. diameter or even larger. Then the sand leakage may become so great as partly to clog the gravel bed within the area.

The author has been reminded many times by filter operators that gravel of about 1-in. diameter often is found on top of finer gravel. There is no mass movement; the gravel is pushed upward one particle at a time. How to prevent gravel from moving in a filter is still the biggest problem in the maintenance of filters.

Condition of Plant Filters

It was assumed that the worst condition would be found in a bed that had received a number of backwashes, and Filter 10 was therefore selected for inspection. Filter 10 was known to have passed some sand in its nearby 11 yr of operation. The filter had been used in high-rate tests and had been backwashed about 8,000 times. This is equivalent to 25–30 years' use in the average filtration plant. After installation of the equipment for handling sand and gravel, the plant was anxious to give the equipment a trial. Removal of the sand from this filter, hydraulically, was started on Dec. 30, 1957.

The sand was removed and the gravel bed inspected and photographed. The actual pumping time was about 8 hr. There were a few shut-downs for changes, and declogging the pipelines. The gravel in the back of the filter was not in very bad condition, as can be seen in Fig. 17a, although there was some mounding, particularly in the foreground. The large gravel (up to about $1\frac{1}{2}$-in. diame-

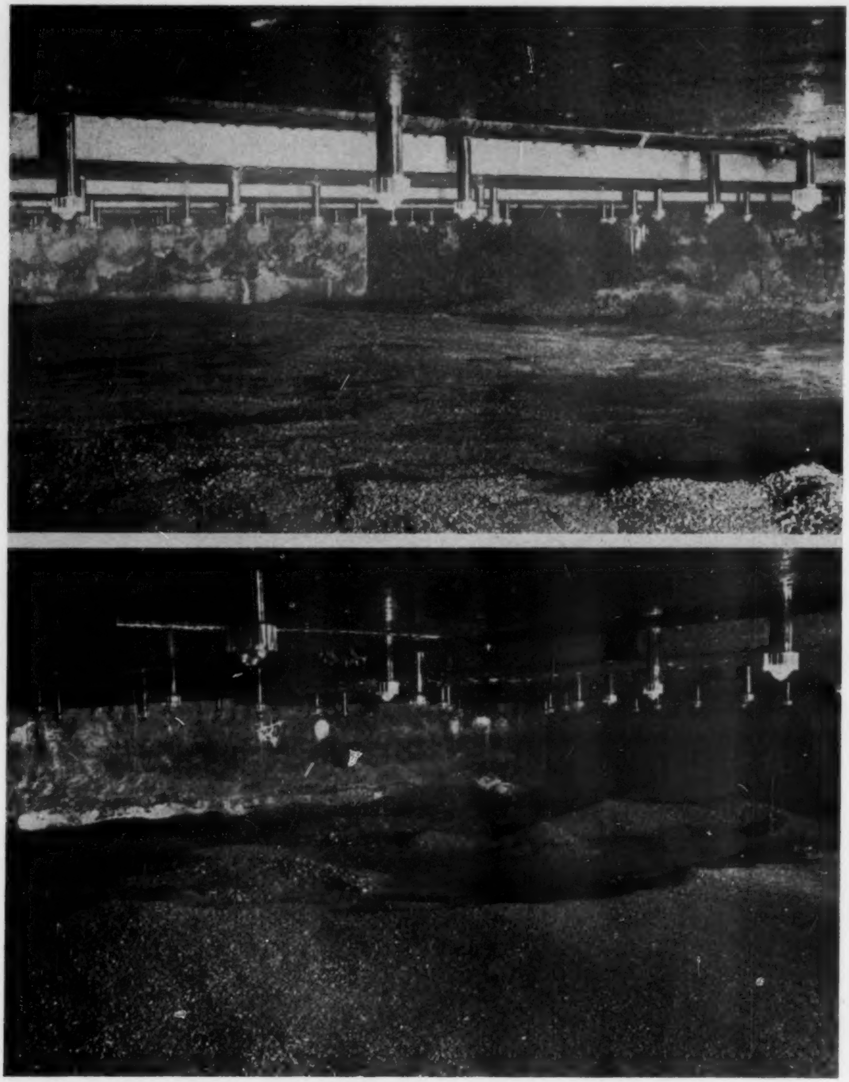

Fig. 17. Filter 10 After Removal of Sand

The upper photograph is of the back end of the filter, where gravel mounding was not very serious. The lower photograph, shows the front end of the filter, where there was more mounding. The large mound in the foreground was 12–15 ft long, and was probably caused by several large boils.

ter) on top of the gravel bed should be noted. The bottom of the wash-water troughs and the surface wash piping are shown in the upper part of the photograph.

The lower part of Fig. 17 shows more mounding at the front end of the filter. Some sand remained on top of the gravel in low places. In a few places there was as much as 3 in. of sand. The mounds would have appeared still higher had all of the sand been removed. No one was allowed to walk on the gravel until it had been photographed. A few particles of gravel over an inch in diameter appeared at the gravel surface. Sand boils were the cause of the displacement of large gravel particles.

The large area of mounded gravel shown in the foreground was probably caused by several large boils, close enough together to create one 12–15-ft mound. The partly circular mound on the right was evidently caused by a large sand boil at this location. Observation of the backwashing before the sand was removed revealed that there was a large sand boil at this location. In fact, there were a number of boils in the area beyond the long, mounded area that is visible in the foreground.

The drawing in Fig. 16 could not have been made without evidence such as was obtained from Filter 10. Ridged or mounded gravel, of course, has occurred in filtration plants all over the country ever since rapid sand filters requiring backwashing began to be used.

For a long time the author thought that the movement of large gravel to the surface of the gravel bed was caused by some unusual disturbance that pushed up large quantities of gravel in spots. It was believed that this could be prevented in filter beds with good underdrainage systems by keeping the sand and gravel clean. This does help, but it has now been revealed that mounding has a different cause.

Gravel mounding is a natural phenomenon, slow of action, that cannot be prevented in any backwash system, regardless of how perfect the underdrains may be in hydraulic design. It is a maintenance problem that filtration plants have met in the past by occasionally rebuilding filter beds. All that can be done in design is to avoid conditions that might aggravate the trouble.

One filter operator recently stated that he believed the solution to traveling gravel was to divide the filter bottom into a number of boxes so the gravel would have to stay within a limited area. When it is observed there is more gravel mounding along vertical walls than in the middle of filter beds, and that gravel particles often shoot upward more than a foot into the expanded bed, it is clear that the partitions would have to extend upward a foot or more into the sand bed, and the idea does not seem a solution. Partitioning gravel beds has been tried in the past, and there must be a reason why it has not come into general use.

Acknowledgment

The South District Filtration Plant is operated by the Department of Water and Sewers. James W. Jardine is the commissioner, W. W. DeBerard is the deputy commissioner and chief water engineer and H. H. Gerstein is the assistant chief water engineer. The author is indebted to sev-

eral plant employees who conducted many of the tests necessary for the preparation of this article. T. D. Nulty, Water Chemical Engineer IV, and Ralph F. Falkenthal, Water Chemical Engineer III, were in charge of the experiments. The photographs are by William Meek.

Reference

1. BAYLIS, JOHN R. A Study of Filtering Materials for Rapid Sand Filters. *Wtr. & Sew. Wks.*, **82**: (Mar. 1935).

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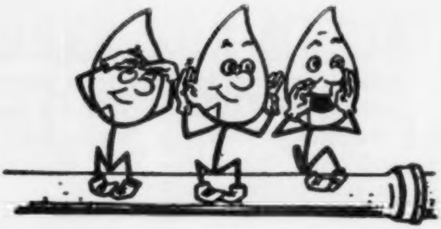

Percolation and Runoff

HAPPY NEW YEAR, we say, with the feeling that it will probably be the "water crisis" that makes it happy. What crisis?

The crisis that led the *Financial Post,* Canada's *Wall Street Journal,* last July to front-page a survey under the question, "Is Canada Running Short of Water?"

The crisis that led the *Saturday Evening Post* in the same month to editorialize the fact that "Water Conservation Is a Top Concern With Many States"

The crisis that led Congress to authorize the expenditure of $10,000,000 on five experimental plant-scale desalinization plants

The crisis that led *Sports Afield* to devote a considerable part of its July issue to a discussion of water rights and problems, headed "The Water Crisis"

The crisis that led President Eisenhower, in his August speech to the UN, to feature—and the newspapers to headline—the fact that desalinization research has put us on "the threshold of solution of the world's water shortage"

The crisis that led the League of Women Voters to adopt water resources as its study project for a second 2-year period

The crisis that led the American Legion to feature a frightening "They Call It—Drinking Water!" in the October issue of its magazine, *Legion*

The crisis that led Interior Secretary Seaton and Health, Education & Welfare Secretary Flemming to make the "ominous water shortage" the major point of their recent public statements

The crisis that led New Jersey voters, after more than 25 years of ducking the issue, to approve funds for additional storage facilities, despite the fact that, for the first time in years, present reservoirs were brimming

The crisis that led voters last November to approve all bonds for water construction projects while they were defeating an overall 32 per cent of the issues before them.

It is in the last two of these items, of course, that the happiness lies, despite the fact that, in helping to lead up to them, the others may have aroused considerable fear and misunderstanding. Meanwhile, it appears to be true that the media of public information are becoming conscious of the full significance of water supply and water resources problems and are taking more pains to put the facts into proper focus. Thus, behind the cries of crisis, behind the semantics of sensationalism, the fact that there is no

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shortage of water, but only of the facilities to make it available, usually come through. And all that remains to bring on not just a happy new year but the millennium is for the public to appreciate this fact.

"All the Water You Need, BUT . . ." is the story. All the water you need, but industries and municipalities must stop making it unusable with their untreated wastes. All the water you need, but you must store it when it is available for the time when you will need it. All the water you need, but you must provide the transmission, treatment, and distribution facilities to make it available in usable form where you need it. All of which makes the "water crisis" merely a "water cri$i$" and your "Happy New Year" as close as your public is not!

Happy Water Works Year it will be in Illinois, where Governor Stratton has just proclaimed:

WHEREAS, Fifty years ago next Feb. 16, a group of 37 water works officials in Illinois, meeting with the Illinois State Water Survey, by invitation of President Edmund J. James of the University of

Governor Stratton prepares to sign "Water Works Year" proclamation in honor of Illinois Section's upcoming 50th anniversary. Looking on are (left to right): H. H. Gerstein, the Section's national director; W. C. Ackermann, chief of the State Water Survey; and T. E. Larson, Section chairman.

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Illinois, formally prepared and adopted the constitution of the Illinois Water Supply Association, and

WHEREAS, This Association affiliated with the American Water Works Association as its Illinois Section in 1914, and

WHEREAS, Water works men from Illinois have been and are actively and conscientiously engaged in developing and using advanced knowledge relating to water supplies and the conservation of water supplies for public use, and

WHEREAS, These men can be justly proud of their contributions toward the future of Illinois, and

WHEREAS, These efforts and achievements should be a matter of public information for all citizens who enjoy the convenience of pure water at their household tap, and

WHEREAS, It is a matter of public interest for citizens and civic leaders to acquaint themselves with and support planned water works development for community growth and well-being,

Now, therefore, I, William G. Stratton, Governor of the State of Illinois, do hereby proclaim the year 1959 as Water Works Year in Illinois, with appreciation to all water works men for their continuing loyal public service, upon this Fiftieth Anniversary date of organization.

High point of the celebration will no doubt be the section's Fiftieth Anniversary Meeting, to be held at the Morrison Hotel in Chicago, Mar. 11–13, but plans have been made to keep the observance going all year through local newspaper publicity, plant visits, and tie-ins with all projects and programs having to do with public water supply.

Having celebrated "Water Works Month" in June and November in Missouri and West Virginia, respectively, and now "Water Works Year," we're looking forward to bigger and better celebrations in the future. Again, the millennium rears its lovely head!

Spaeder from Decatur—Harold J. Spaeder, that is, from Decatur, Ga.—seems to have been 1958's Anonymost. So much so, in fact, that though he was to have appeared as '58's 58th member of Editors Anonymous (see December P&R, p. 35), his innominacy was entirely unintentionally completely preserved. In apologizing now for allowing him to go scot-free, we salute him as an $EA^3 \rightarrow EA^4_{2b}$.

Tank traps are being set again for unwary owners of elevated steel water storage tanks. Within recent months one community in California and another in Kentucky entered into tank-painting agreements which permitted the contractor to inspect and make necessary repairs to the tank before painting. In California a suit resulted; in Kentucky the community was able to avoid a repair bill more than six times the amount of the painting contract by calling in an independent inspector, and then was lucky enough to find a new painter in a hurry when the original crew walked off the job. But both the cost of the suit and the risk of prolonged loss of storage could have been avoided by such precautions as preinspection and the separation of painting and repair contracts.

It was to help water works men protect themselves in the handling of these and other details of painting contracts that AWWA more than 10 years ago worked out standards for inspecting, repairing, painting, and repainting steel tanks, standpipes, reservoirs, and elevated tanks, for water storage (D101

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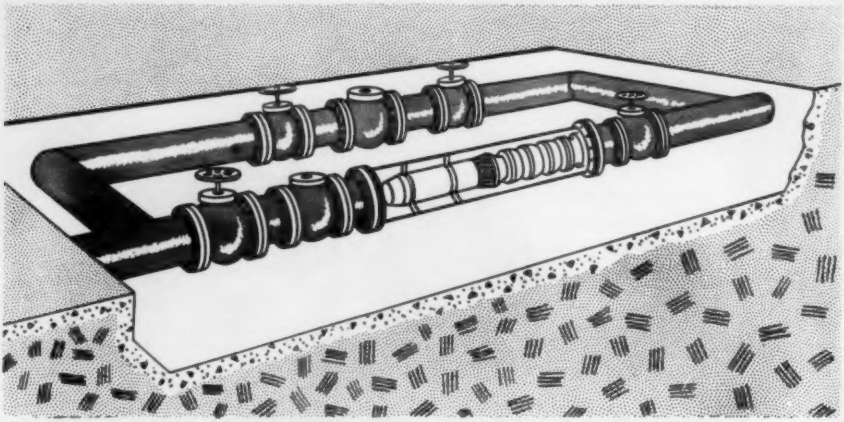
& D102, combined, 50 cents). These standards, have already saved tank owners millions of rivets, dollars, and headaches. If your tank needs paint, they're worth getting before letting.

A submersible booster pump designed to operate in a horizontal position within the pipeline itself is being marketed by Layne & Bowler Pump Co., Los Angeles. Named the "Power-Line," the pump can be suspended within a special section of the pipeline



(see cut). This section is flanged at each end and becomes an integral part of the line. The motor is centered and firmly held by special spiders within the section. The submersible power cable leads to a terminal box mounted on the outside of the section and thence to a control box. Available in sizes designed to operate within pipe from 8 to 24 in., capacities from 100 to 4,000 gpm at heads of 50–500 ft are claimed for the pump.

S. Morgan Smith Co., York, Pa., announces its board of directors has approved a plan, subject to ratification by the stockholders, whereby Allis-Chalmers Mfg. Co., Milwaukee, Wis., would acquire the assets of Smith in exchange for Allis-Chalmers common stock. It is the intention of the two firms to conclude the transaction as soon as possible.

Walter E. Jessup has retired as editor of *Civil Engineering,* official ASCE publication. His successor is Hal W. Hunt, who has been executive editor since 1957. Mr. Jessup has been with ASCE for 28 years. He was the first editor of *Civil Engineering* (which was established in 1930), a post he held until 1935, when he became the society's field secretary. He resumed the editorship in 1948, after serving in the Army and as western representative for ASCE. Mr. Hunt has had a wide variety of construction experience and is a former associate editor of *Engineering News-Record.*

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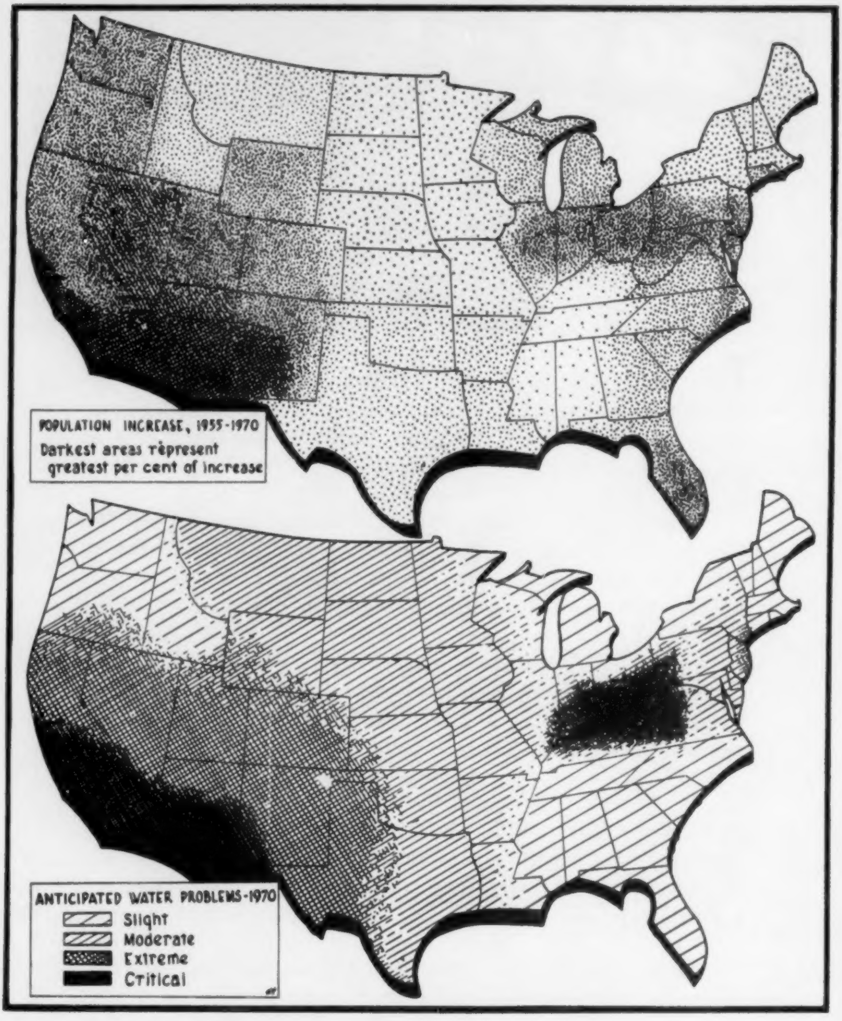

Population and Water Supply—1970

The upper map shows the distribution of population in 1970, as projected by the US Bureau of the Census; areas with anticipated water shortages, as forecast from projections of US Dept. of Commerce and USGS data, are indicated on the lower map. The information was compiled by the Population Reference Bureau, Washington, D.C.

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ASEIB—the American Sanitary Engineering Intersociety Board—has accepted 100 additional engineers for certification. All certified engineers automatically become diplomates in the American Academy of Sanitary Engineers. The total number of diplomates is now 979. Of those previously certified, 555 were qualified through specialization in water supply and waste water disposal, 198 in public health, 31 in industrial hygiene, 10 in air pollution control, and 5 in radiation hygiene and hazard control. An additional 80 had qualified in the general field of sanitary engineering.

At its recent meeting, the board reelected Thomas R. Camp chairman, R. E. Lawrence vice-chairman, and R. S. Rankin treasurer. Mr. Rankin was also elected secretary, replacing Francis B. Elder. Reelected as trustees, with terms expiring in 1961, were W. L. Faith, Raymond J. Faust, J. E. Kiker Jr., R. S. Rankin, and Clarence I. Sterling Jr. B. A. Poole replaced W. A. Hardenbergh. Mr. Faust was reappointed chairman of the Specialty Committee.

Special action taken included authorization of publication of the first roster of diplomates in January 1959. A much debated question regarding the specialty listings to appear on certificates was resolved by authorizing issuance of certificates bearing the specialty designation, "sanitary engineer." Anyone previously issued a certificate showing another specialty will be allowed to exchange it free

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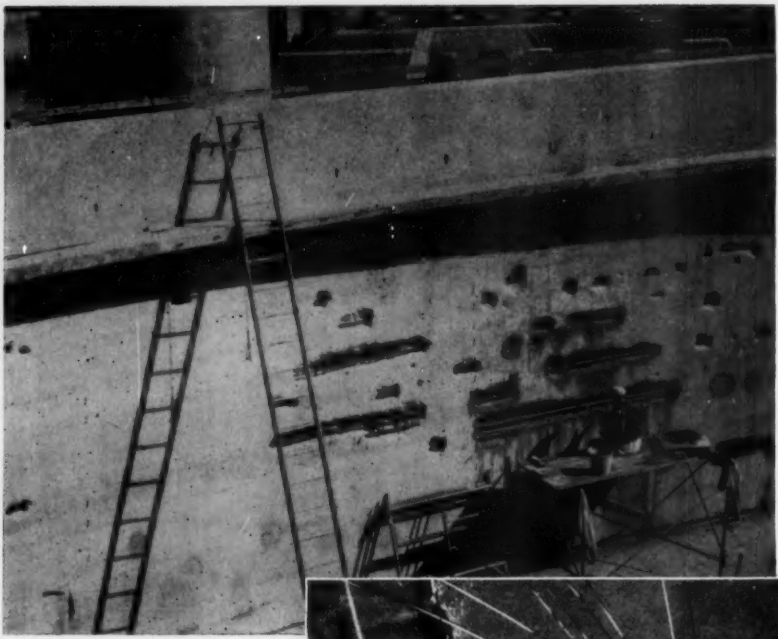
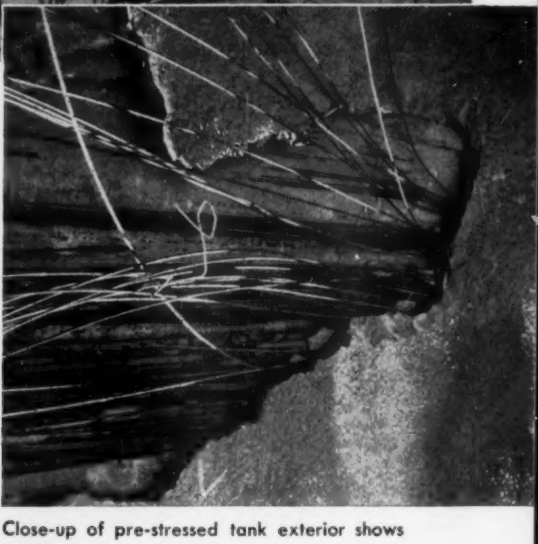
CONCRETE NEEDS CORROSION



Damage caused by sub-surface corrosion of steel reinforcing rods is shown in this view of tank interior.



Close-up of pre-stressed tank exterior shows large sections of gunite which were forced from wall by corroding wires.

(*Continued from page 44 P&R*)

of charge for one inscribed sanitary engineer.

Letters of commendation were prepared for two retiring members of the board—for Col. Hardenbergh, in recognition of his many years of contributions culminating in the establishment of the ASEIB and the academy; and for Mr. Elder, for his work as secretary during the first 3 years of the organization.

Stranger than friction are its cures, it would seem, if the story which recently appeared in *The Extinguisher*, a fire insurance magazine, has any basis in fact. The story, in full:

A chemical solution said to be capable of reducing friction losses in water mains is presently undergoing extensive field tests in the Victor Village of Stanford, N.Y. Minute quantities of the solution introduced at any point in a water supply system are reported to reduce friction loss in pipe and fittings by approximately 85 per cent. It is reported that the small quantity necessary will have no effect on the potability of the water supply.

By whom reported or on what basis we have been unable yet to determine. The state health department has been unable to trace the tester or even his last testing place, indicating, only, that whatever it is, it's illegal. The insurance agency assures us that we have all the information it has, but that if whatever it is is what it is reported to be, it's welcome. All of which leaves us to speculate on whether it

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is a purgative, a lubricant, or just another illusion. Whatever it is, it should certainly be named "High *C*'s," which spelled elsewise is probably where the inventor is right now. Or could he have been the same man as invented that new deodorant, "Vanish" (rub a little on and you disappear, leaving people to wonder where the smell comes from)? He might be conducting "extensive field tests" of that simultaneously. At any rate, *something* smells.

P.S.: Smell will tell—the facts, finally unearthed by our friends in the Factory Mutual Engineering Div. after we had committed ourselves in the above terms, are that the story itself was a hoax, aimed at measuring reader response. Hmmmmmm!

Clark, Daily & Dietz, consulting engineers, announce the addition of W. D. Painter as a partner in the firm, which has opened a new office in Carlyle, Ill.

Walter K. Davies has been appointed foreign sales coordinator for Neptune Meter Co. and will supervise all activities relating to product sales outside the United States by Neptune and its six subsidiaries. Mr. Davies has been with the firm since 1954.

Weather control, once our favorite subject, isn't really dead—it's just over our head these days. The best—or, at least, most understandable—indication of its liveness was a comprehensive survey of the field in *Fortune* magazine last May, wherein not only rainmaking and storm suppression, but lightning control and long-range weather forecasting were discussed,

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with the conclusion, "Forecast for Weather Control: Brighter." What has kept us still is that so little new that even we would be willing to call conclusive has been happening since we reviewed the use of weather as a weapon eight months ago (May 1958 P&R, p. 44). Since then new radar equipment capable of detecting raindrops 50–100 mi up in the sky has come upon the scene. The computers have been put to work in an effort to establish trends and patterns and bases and predictability equations from which experimental results can be judged and new directions of research determined. Meanwhile our satellites have been signaling back information that may directly link solar events to changes in our weather and, thereby, presumably, start us tinkering with the sun. And throughout it all, anti statisticians have been laughing at pro statisticians and anti meteorologists have been deriding pro meteorologists and anti scientists have been discrediting pro scientists until, finding ourself more anti-anti than pro-pro, we've stepped aside to wait and see—the only way to be really sure about weather anyway!

The reason we did bring the subject up, though, was to report that Hatfield the Rainmaker, the man who was paid $1,000 an inch on rainmaking contracts in the US and Italy beginning as long ago as 1904, died at age 82 early in 1958 in Pearblossom, Calif., carrying with him the secret of how he broke the drought in San Diego in 1916 with 16 in. of rain in a single day, part of a storm that poured 40 in. on the area, filling the city's reservoir, knocking out two dams, flooding ranches and homes, and doing millions of dollars of damage. Hatfield, as far as we are concerned, was the real McCoy—a man who was seeding the clouds by the use of a chemical burner long before most of the current crop of seeders were even seeds.

Meanwhile, one further event of possible significance in the field was the development, by Dr. Florence W. van Stratten, a Navy meteorologist, of a technique for creating clouds. Hitherto, rainmakers have been hampered by the necessity of waiting for clouds to seed. Now if Dr. van Stratten's technique of dropping liquid suspensions of carbon black from a plane can indeed create the necessary seedlings, we shouldn't have long to wait to see.

Forsooth, perhaps for soot if not for snow!

Roberts Filter Mfg. Co., Darby, Pa., announces the appointment of Jesse W. Roberts as vice-president and John W. Burton as general manager. Both men have been with the firm for many years.

(*Continued on page 86 P&R*)

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Key: In the reference to the publication in which the abstracted article appears, **39**:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, **39**:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

FOREIGN WATER SUPPLIES —GENERAL

Planning of Large-Scale Water Supplies in Austria. H. Suritsch. Öst. Wasserw. (Vienna), **8**:81 ('56). Rainfall records and records of flow of Danube at Vienna from '01 to '50 show no decrease in available water. Demand, however, has greatly increased owing to growth of pop. and industry and increased individual demand. Diagram showing increases in pop. and demand in Vienna, Graz, and Linz is given. Author then discusses distr. of pop. and of sources of water supply in Austria and possible development of water works serving several communities or large dists. There are, in Austria, 30 plants serving 2 or more communities. Geographical and economic considerations affecting development of such supplies are considered and importance of large-scale planning and cooperation is emphasized.—*WPA*

Some Recent Works in Baghdad. G. Smethurst. J. Inst. Water Engrs. (London), **10**:123 ('56). General description of Iraq is given, followed by historical account of development of Baghdad, special attention being paid to water supply of city. Baghdad District Water Board was formed in '23 to modernize water supply system, but it was not until '48 that city could be said to have adequate water supply. In most parts of Iraq, ground water is present but contains high concns. of sulfate, while in Baghdad itself it is pold. by leakage from 50,000 cesspools, as city has, as yet, no sewerage system. River water is, therefore, used as source of water supply. In '48, board had 3 water treatment plants situated at Sarrafiyah, Kerrada, and Shalchiyah, with capacs. of 12, 3, and 3 mgd resp. Owing to growth of city, it was decided that Kerrada and Shalchiyah plants should be extended, former to capac. of 6 mgd, and latter to 13 mgd. Detailed descriptions of both plants and their construction are given. At Kerrada plant, water is treated by addn. of sulfate of alumina, primary and secondary sedimentation, filtration through horizontal rapid pressure filters, and chlorination. Dose of 0.5 ppm chlorine is used. At Shalchiyah plant, treatment comprises addn. of aluminium sulfate, primary and secondary sedimentation in horizontal-flow tanks, and filtration. Flow diagrams for both plants are included. Further work has also been carried out in areas outside Baghdad on filters to provide local water supplies for drinking purposes and swimming pools, on constr. of floating water works, and on constr. of water supply system for Dujaila village. Article includes many photographs and diagrams and details of costs of constr. of projects described, and is concluded by discussion. —*WPA*

A Survey of Additional Sources of Supply for Liverpool. J. Br. W. W. Assn. (Br.), **38**:101 ('56). Survey is given of need for addnl. sources of water supply for Liverpool, of various alternatives which have been considered and, finally, recommendations adopted by Liverpool Corp. Water Committee in order to solve problem. Statistics are given which show that in yr ended Mar. 31, '55, demands for water were met by drawing from Rivington works supplies in excess of reliable yield. To augment supplies consideration has been given to possible sources in N. Wales and Lake Dist., and prospect of bulk supply from Manchester Corp. In order to meet immediate needs it is proposed to seek approval to reduction of compensation waters from Vyrnwy and Rivington works, and to obtain bulk supply—at present not available—from Manchester in 2 or 3 yr. It is anticipated that these increments will be fully utilized in about 8 yr when addnl. supply will become available from recommended Tryweryn, N.

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(*Continued from page 62 P&R*)

Wales, scheme with yield of 70-80 mgd.—*WPA*

The Problem of the Water Supply of Built-Up Areas in Morocco Under French Protectorate. Kabst. Aqua, 4:6 ('56). Central planning of water supply system is necessary in Morocco for financial and technical reasons. New schemes take acct. of increasing pop. and need to raise present low rate of supply per person. In '54, tot. of 18 bil gal of water was supplied for pop. of about 2,000,000 people. Progress has been made possible by technical advances in design of mains, pumps, valves, and reservoirs, and in purification of water. Recent work includes increased water supplies for Casablanca from Oum-er-Rebia, for Safi from Ain-Rhor and Ain R/Tem, and for Meknes from the Ain Bittit and Ain Ribaa. New catchment areas at Sidi Taibi and Si Ahmed Thaleb have increased Souarat supply, dam has been built on Wadi Zamrine to supply Khouribga, and wells have been provided for Djerada and Marrakech. Acct. is given of water supply systems developed in Morocco from Roman times up to time of arrival of French.—*WPA*

General Water Supply Planning for the Rio Dulce (Arg.). F. M. Kupfer. Wasserwirtschaft (Stuttgart), 47:86 ('57). Author gives illustrated description of water supply conditions in N. Arg., especially drainage area and valley of R. Dulce. Area is subtropical, rainfall is concd. in 3 summer mos., there are frequent dry yr, and much potentially valuable agricultural land is wasted for lack of water. Problems include storage of summer rainfall and equilization of flow over wet and dry yr. Investigations made and work done and planned are described.—*WPA*

Rod el Farag Water Works for Nile Water Treatment. H. Suleiman. Aqua, 3:1 ('56). Author describes treatment of water from R. Nile at Rod el Farag water works, Egypt. Nile shows abundance of diatoms

(*Continued on page 66 P&R*)

(*Continued from page 62 P&R*)

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Rod el Farag Water Works for Nile Water Treatment. H. SULEIMAN. Aqua, 3:3 ('56). Author describes treatment of water from R. Nile at Rod el Farag water works, Egypt. Nile shows abundance of diatoms

(*Continued on page 66 P&R*)

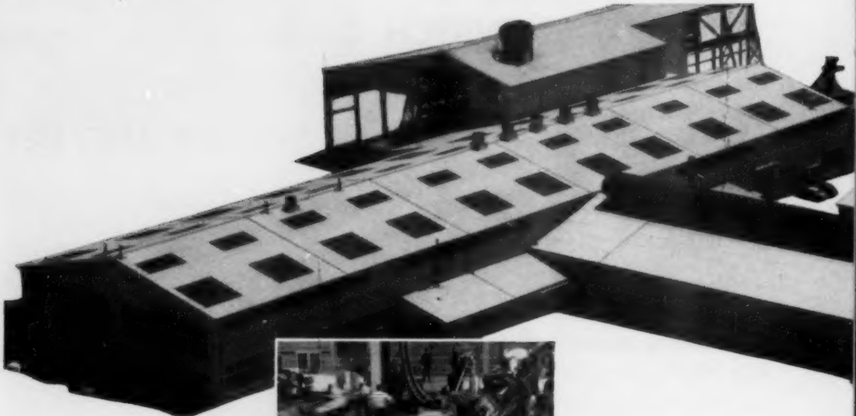
(*Continued from page 64 P&R*)

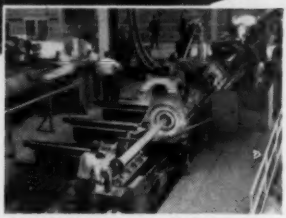
for 2–3 wk in Feb., Mar., or Apr. These are odorless. In Jun. and Jul., large numbers of *Anaboena* and *Spirogyra* appear; these have unpleasant odor and cause muddy taste in water. During flood period in Sep., turbidity of water is increased greatly by suspended particles of argillaceous matter and by increase in colloidal matter. Water works consists of 2 sections. South section comprises 13 sedimentation tanks, and north section 3 clarifiers. Aluminium sulfate is used as coagulant and, during periods of high turbidity, lime is also added. —*WPA*

Woodland and Water Supply in the Ruhr District. E. KIRWALD. Gas- u. Wasserfach (Munich), **97**:489 ('56). Brief report is given of results of observations at 7 stations on streams in Ruhr dist. mainly in woodland. Observations reported cover 3 yr ('51–'53); work is being continued and full acct. will be published. Figures are given and discussed for rainfall, runoff, stream flow, seepage, and effects of dry periods, retention, and flow-off of seepage water, melting snow, variations in temp., condition of soil, and formation of drainage area are discussed.—*WPA*

Corporation of Weston-super-Mare Water Undertaking. J. Br. W. W. Assn., **39**:178 ('57). Since '23, principal source of water for Weston-super-Mare has been Banwell spring. In summer, to meet increased demand during holiday season, this was augmented with water from shallow well, original source of supply. This water is, however, very hard. Neither of these sources requires treatment other than disinfection. Peak holiday demand in summer is 2¼ mgd and, as result of falling yield of Banwell spring, this could no longer be met. Agreement was reached with Bristol Waterworks Corp. to receive max. of 2 mgd of raw water from Cheddar Res., and that necessary treatment plant should be completed early in '58. Early in '56, it appeared that deficiency of water in summer would be greater than usual and it was decided that completion of raw water main from Cheddar Res. should be hastened and temporary portable filters installed to treat water. Portable "Stellar" diatomaceous-

(*Continued on page 70 P&R*)





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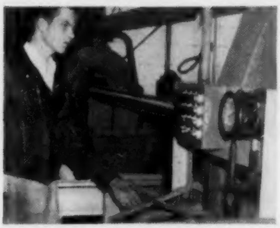

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(*Continued from page 66 P&R*)

earth candle filters were installed and proved satisfactory, although presence of algae in raw water reduced rate of filtration considerably. Water was then chlorinated before being mixed with spring water.—*WPA*

The City of Worcester Water Undertaking. A Review. J. Br. W. W. Assn., **39**:192 ('57). Historical acct. is given of development of water supply for city of Worcester (Br.). Water is obtained from R. Severn and is now treated by sedimentation, rapid gravity filtration, and slow sand filtration. Recent installation of rapid gravity filters has improved bact. qual. of water and lengthened filter runs of slow sand filters. —*WPA*

Gothenburg Water Works and the Salt Sea-Water. G. Blidberg. Aqua, **1**:16 ('57). Water supply for Gothenburg, Swed., is obtained from Goteborg branch of Gota Alv, one of largest rivers in Scandinavia. Treatment plant is at Alelyckan, 12 km (7.5 mi) from sea. In recent yr, after intensive dredging operations, salt water has extended upriver to this point during periods of low flow, and has entered treatment plant. Measures taken to increase flow in this branch of river, and to regulate L. Vanern, further upstream, where water is used for power stations, are described. Plans to increase depth of river still further by dredging will adversely affect salinity of water at site of intake, and it will then probably be necessary either to move intake further upriver or to select new source of water, such as L. Vanern.—*WPA*

The Solution of the Drinking-Water Problem in the Rural Centers of Vietnam. A. V. Goudou & C. Richard. Aqua, **2**:8, 12 ('57). Raw water for rural centers of Vietnam is obtained from Mekong and Red rivers or their tributaries. During dry season these waters are brackish. Artificial ponds have been built to store fresh water during rainy season, for use during brackish season. Purif. is effected by flocculation using aluminium sulfate as coagulant, and rapid filtration through gravel. Clarified water is chlorinated by a concd. soln. of chloride of lime. Simple plants, with capac. of 10 cu m/hr, have been established all over country. Diagrams of typical plant are included.—*WPA*

TASTE AND ODOR

The Causes of Tastes and Odors in Drinking Water. R. D. Hoak. Proc. 11th Ind. Waste Conf., **91**:229 ('56). Presence of phenolic tastes and odors in water supplies has usually been attributed to trade waste waters, but recent studies have shown that phenols may occur from natural sources. Some bacteria produce phenols as metabolic byproducts and certain aquatic plants release phenols. Some lab. expts. made to obtain quant. data on production of phenolic materials by decomposing vegetation are described. Expts. have also been carried out to study reaction between phenol and chlorine and to establish conditions under which chlorophenol odors occur. They showed that phenol does not react with chlorine at low pH values, that chlorophenol taste may occur in distd. water with marginal chlorination at pH values above 7.7, and that phenol can be decomposed into tasteless byproducts at pH values above 7.7 provided that at least stoichiometric amt. of chlorine is added and that sufficiently long reaction period is allowed. It is pointed out that many of conclusions that have been reached on reaction between phenolic substances and chlorine have been based on lab. expts. using pure compds. and that different results may be obtained when reaction is investigated in field. (Wtr. & Sew. Wks., **104**:243, '57.)—*WPA*

Odor Development in the Chlorine Dioxide Treatment of Waters Containing Phenols. J. Holluta & K. Haberer. Gas- u. Wasserfach. (Munich), **98**:552 ('57). Waters contg. phenols develop objectionable tastes and odors when treated with Cl; ClO_2 eral odor max. also occur with ClO_2; exact molecular ratio of ClO_2:phenol for these varies with phenol content. In practice min. odor is developed with water contg. 0.1 mg/l, when ClO_2:phenol molecular ratio is 8:1, corresponding to wt. ratio of 5.7:1. —*CA*

Odor Counteraction. R. H. Wright. Chem. in Can. (Ottawa), **10**:4:37 ('58). Arguments for phenomenon of odor counteraction (**I**) were discussed according to current knowledge. It was concluded that there is no conclusive evidence for **I**.—*CA*

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Preliminary Observations on How to Weaken or Eliminate the Earthy Odor of River Water. V. I. USPENSKAYA. Byull. Moskov. Obshchestva Ispytalel Prirody, Otdel Biol. (Moscow), **62**:1:43 ('57). Algae, *e.g., Spirogyra crassa, Cladophora fracta, Scenedesmus quadricauda,* species of *Coelosphaerium, Merismopedia, Pediastrum, Closterium,* and *Cosmarium,* and diatoms, *e.g., Navicula, Amphora ovalis, Melosira varians,* and *Fragilaria crotonensis,* were grown in culture and then such cultures were submerged in rivers to be treated, in sufficient amts. that cultures (pure or mixed) could continue to grow. Important thing is that algae are used which naturally do not occur in water to be treated. Once growth has started nicely improvement, even occasionally until complete abatement, of odor can be noticed. It is shown that O in *status nascendi* developed by such alga is agent responsible for this effect. It might become necessary to stir water so as to aerate algae, that they can grow and develop enough chlorophyll, before they can show photosynthesis with subsequent development of nascent O.—*CA*

Water Treatment for the Eradication of Tastes and Odors. G. CARTER. Proc. Soc. Water Treatment and Examn., **6**:43 ('57). Effects of common treatment methods on taste and odor are discussed. Prevention through water management and knowledge of pollutional dischgs. are aid in control. Most tastes and odors can be handled by aeration, chlorination, ozone, and carbon. Slow sand filtration is also efficient in removing tastes from stored water. Cucumber-taste episode believed caused by *Synura Uvella* resisted all attempts at control. Chlorophenol tastes are among worst encountered. Combination of chlorine with iron bact. has given rise to chlorophenol tastes. Musty tastes and odors developing in distr. systems are believed to come from organisms living on organic debris in pipes. —*PHEA*

(*Continued on page 74 P&R*)

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Tastes and Odors in Water—A Historical Introduction. F. C. BULLOCK. Proc. Soc. Water Treatment and Examn., **6**:29 ('57). Author discusses rather cloudy history of taste and odor in water. In addn., he gives some case histories of taste and odor problems which have been caused by everything from cross-connections to dead swallows in hot-water feed tank.—*PHEA*

Insuring a Palatable Supply for Des Moines. J. R. MALONEY. Taste Odor Control J., **22**:2:1 ('56). Water supply for Des Moines, Iowa, was formerly obtained from infiltration gallery in glacial sands of Raccoon R. valley, but to meet increased demand after WW II facilities for direct intake of river water into system were incorporated into design of new water-softening plant. In '54 expts. were made on use of activated carbon to control tastes and odors in river water to required threshold odor value of 2. Results have so far been satisfactory and lab. expts. have indicated that alum coagulation and/or treatment with activated carbon reduces chlorine demand of water. Flow diagram of plant is given.—*WPA*

The Nature of Odor in Water. A. M. ARENSCHTEJN. Gigena i Sanit. (Moscow), **21**:3:45 ('56). Investigation is described into connection between odor in water and microorganisms present. Ether extracts of water, water organisms, and water plants were examd. organoleptically and by luminescence anal. under ultraviolet light. Water plants and organisms had specific odor and specific luminescence. Pure cultures of microorganisms were also examd. and luminescence colors of no. of these are given. Both luminescence and odor are reduced by treatment of water.—*WPA*

Hackensack Water Company—A Quarter Century of Water Purification. P. E. PALLO & P. TAMER. Taste Odor Control J., **22**:5:2 ('56). Authors describe developments over last 25 yr in use of activated carbon at purif. plant at Hackensack Water Co., Oradell, N.J., for control of tastes and odors in water supply.—*WPA*

Taste and Odor Experience at Rockingham, N.C. E. R. TULL. Taste Odor Control J., **22**:8:1 ('56). Water for Rockingham, N.C., comes from creek flowing through swampy land to shallow impounding res. with heavy aquatic weed growth. Water treatment consists of addn. of alum and lime, sedimentation, rapid sand filtration, postchlorination, addn. of lime and metaphosphate for adjustment of pH value, and fluoridation with sodium fluoride. Carbon is applied at top of filters as safety factor. During period of drought in '53, when water level in res. was very low, sudden production of brown water with repulsive taste occurred. Immediately amt. of carbon added to filters was increased, plant was obtained to give prelim. chlorination, and continual lab. tests were made in order to find cause of trouble. Drastic treatment only just maintained palatable water. 3-in. rainfall brought conditions suddenly back to normal and cause of color, taste, and odor formation was never definitely found, although it is thought to have been due to actinomycetes, feeding on nitrogenous matter in res.—*WPA*

Taste and Odor Control . . . Murfreesboro, Tennessee. E. JOHNSON. Taste Odor Control J., **24**:1:1 ('58). Murfreesboro, Tenn., originally depended upon Murfree Spring and no. of deep wells for its water supply, but these sources became inadequate and since '45 water has been drawn from Stones R. to supplement original supply during summer mos. Present treatment consists of prelim. chlorination, coagulation with alum, softening with lime, carbonation, sedimentation, rapid sand filtration, and postchlorination. Raw water from both sources is contamd. by tastes and odors. Activated carbon is now used to elim. tastes and odors and point of application is varied according to season of yr. Flow diagram of water treatment plant is included.—*WPA*

Types of Taste and Odor and Their Estimation. E. W. TAYLOR. Proc. Soc. Water Treatment Examn., **6**:39 ('57). Author classifies various types of tastes and odors which may occur in water supplies, together with their causes and describes method of detecting tastes and odors used by London Metropolitan Water Bd., and that recommended in Am. Standard Methods. In Am. method, water is smelled without tasting, whereas in Metropolitan Water Bd. method it is tasted, thus permitting detection of tastes without odor, such as brackish tastes.—*WPA*

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WELLS AND GROUND WATER

Pecularities of the Chemical Composition of Ground Waters of Soil of the River Flood Plain and Factors Determining Them. A. N. TYURYUKANOV. Pochnovedenie (Moscow), **9**:79 ('57). Seasonal changes of chem. compn. of ground waters of bottom land of lower part of Moscow R. have been examd., together with changes of BOD. It was found that chem. compn. of soil-ground waters is detd. mostly by processes of soil formation, by compn. of silt deposits of bottom lands, and chem. compn. of river H_2O. Max. mineralization of soil and of ground waters is observed in spring when there are floods (that is silt stage of bottom-land soil formation), and one finds increase of concns. of NH_4^+, Ca^{++}, Mg^{++}, Fe^{++}, SO_4^{--}, and HCO_3^-. Main characteristics are given of soils of river valley land of Moscow R., i.e., of meadow soils, sod-meadow soils, meadow-boggy soils, and slightly podzolized soils of 1st terrace above bottom land.—*CA*

Experimental Investigations on Ground Water in Germany. C. R. BAIER. Bull. centre belge etude et document. eaux (Liege), **31**:47 ('56). Description is given of work carried out to det. availability of ground water as source of water supply in Ger. Availability of ground water resources was detd. and evaluated; and importance of their natural replenishment by atmospheric pptn., possibilities of artificial enrichment of ground water by surface water, and mechanism of infiltration of pold. surface waters, waste waters, and rinsing waters into subsoil, were investigated. Some results of these investigations are given in tables and graphs. Lysimeters were used extensively during investigations, and diagram of lysimetric installations at Dortmund (W. Ger.) is included.—*WPA*

Ground-Water Resources of the Northeastern Part of Volusia County, Florida. G. G. WYRICK & W. P. LEUTZE. Florida Geol. Survey, Inform. Circ. No. **8**:1 ('56). Chem. anals. are given of 43 samples of well

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waters. Detns. of chloride in 1,300 samples show that some encroachment of salt water has occurred in 3 coastal areas; it may be avoided by developing wells in areas where salt water lies at considerable depth or by avoiding large draw-downs.—*CA*

Ground-Water Geology of the Bruneau-Grand View Area, Owyhee County, Idaho. R. T. LITTLETON & E. G. CROSTHWAITE. U. S. Geol. Survey Water Supply Paper No. **1460-D**:147 ('57). Area is largely within Snake R. valley, and several artesian aquifers occur. Ground water contains F, locally above 20 ppm; those in N. are high in Na and unsuitable for irrig. Widespread irrig. from artesian water is feasible in most parts, provided adequate soil drainage and soil amendment are maintained.—*CA*

Geology and Ground-Water Resources of the Lower Marias Irrigation Project, Montana. F. A. SWENSON & H. A. SWENSON. U.S. Geol. Survey Water Supply Paper No. **1460-B**:41 ('57). 2 submerged sandstone (Virgelle and Judith R.) aquifers and several alluvial fans constitute ground water supply of area. Virgelle aquifer is highly mineralized (Cl^-) and unsuitable for irrig.; but water from Judith R. aquifer is of good qual. Irrig. from ground water or planned surface reservoir water must be carefully controlled to avoid waterlogging in area.—*CA*

Geology and Ground-Water Hydrology of the Valleys of the Republican and Frenchman Rivers, Nebraska. E. BRADLEY & C. R. JOHNSON. U.S. Geol. Survey Water Supply Paper No. **1360-H**:589 ('57). Area consists of 370 sq m of flat or gently sloping land bordering rivers on either side. Large quants. of water can be obtained from Quaternary sands and gravels that fill natural storage troughs in Cretaceous bedrock lying roughly parallel to valley axes. Water is predominantly Ca Mg bicarbonate, except for western part of area where Ca and Mg sulfates predominate. Water is suitable for domestic, agricultural, and most industrial purposes.—*CA*

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Employment Information

Classified ads will be accepted only for "Positions Available" or "Position Wanted." Rate: $1.50 per line (minimum $5.00), payable before publication. Deadline for ad copy: first of month prior to month of publication desired. To place ad, obtain "Classified Ad Authorization Form" from: Classified Ad Dept., Journal American Water Works Assn., 2 Park Ave., New York 16, N.Y.

Positions Available

(Continued from page 78 P&R)

Salt Balance in Ground Water Reservoir Operation. D. B. Willets & C. A. McCullough. Proc. ASCE, **83**:IR2:No. 1359 ('57). Development of water in Calif. to meet ultimate water requirements of state will require planned utilization for seasonal and cyclic storage of many ground water reservoirs available. Important problem in such operation is maint. of suitable mineral qual. of ground water. Authors discuss sources and disposal of salts in ground water reservoirs and indicate salt-balance requirements for operation of hypothetical ground water reservoir. It is stressed that present knowledge is inadequate, and much more data are required to predict and evaluate salt-balance requirements for reservoirs under planned operation.—*WPA*

Total Analyses of Several Turkish Well Waters. H. Keskin & S. Yanco. Rev. fac. sci. univ. Istanbul (Turk.), **21**:126 ('56). Anals. of well waters of 14 different communities are given, with brief directions for detns. For F, known amt. of monosodium phenyl phosphate (I) is decomposed by potato phosphatase, and F-content is calcd. from PhOH liberated. Soln. of I is fermented with phosphatase, and free PhOH detd. F was detd. graphically from curve of F concn. vs. % decompn. of I. F (in γ) = 0.1, 0.2, 0.3, 0.4, and 0.5; decompn. of I (in %) = 93.75, 89.7, 84.5, 79.5, and 7.8, resp. F in mg/l varied from 0.005 in water contg. little Fe to 0.165 in water contg. 43.0 mg Cl/l.—*PHEA*

Wells Drilled in the Belgian Congo by Injection of Water. G. Borgniez. L'Eau (Paris), **43**:31, 87 ('56). Recent sand deposits in Belg. Congo influence method used for excavation of wells; method employed is by injection of water and basic theory involved is summarized. Theory of well water supply is reviewed and expression for detn. of yield of well is given. Std. method for detg. coef. of permeability for sand is described and improvements on this method are outlined. Possible effects of ground water on materials used in well, methods of preventing them, and choice of suitable materials for constr. of well and equip., are discussed.—*WPA*

Service Lines

Reduced-voltage starters, designed for pumps, conveyors, and other equipment, are described in a new 12-page booklet that includes illustrations and a pocket on the inside cover for insertion of individual data sheets. Copies of Bulletin 14B8192 are available on request from Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.

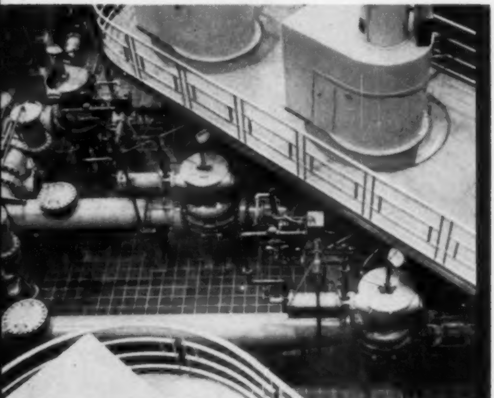
Stream analyzers are the subject of an 8-page, two-color catalog, which includes photographs and information relating to pH equipment, flow colorimeters, and dissolved-oxygen analyzers. Bulletin CL-4000 is available from Beckman Scientific & Process Instruments Div., 2500 Fullerton Rd., Fullerton, Calif.

A pocket-size slide rule calculator, for determining capacities of tanks in gallons, barrels, and pounds of water, may be obtained from Hammond Iron Works, Warren, Pa.

Throats for differential producers can be easily selected with the aid of a 12-page bulletin recently published. Made up largely of equations, performance charts, and capacity tables, the booklet is designed to assist in determining throat size and head loss in various applications of flow tubes, Venturi tubes, and Venturi nozzles. Technical Bulletin 100-P6 may be obtained from Builders Providence, Inc., 345 Harris Ave., Providence 1, R.I.

Ion exchangers are described in a new 8-page, illustrated booklet, which emphasizes the hydrogen base exchange process. Bulletin 4530-A is available by writing Cochrane Corp., Philadelphia 32, Pa.

Washington, D. C., Arlington County, Virginia and parts of Fairfax County, Virginia have unique, efficient, and progressive means of providing more effective water service for public health, convenience and fire protection. The Supply System was designed, built and is operated by the Corps of Engineers, United States Army. The Distribution System in the national capital is operated by The Commissioners, District of Columbia, Department of Sanitary Engineering. More than one million persons are being served and they are being served well . . . 153,157,000 gallons per day.

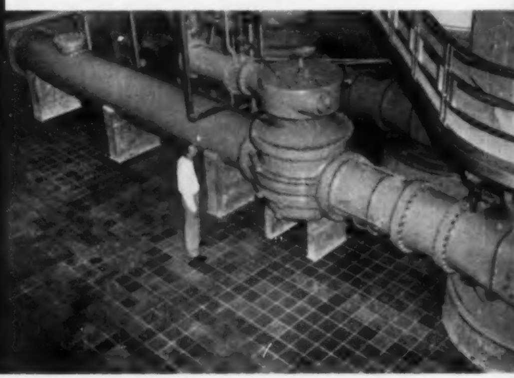


Interior views of the completely air-conditioned Dalecarlia Pumping Station showing some of the many Chapman Cone Type Valves used largely as protection from reversal on power failure of pumps and piping. The Dalecarlia Hydroelectric Station utilizes surplus water for generation of electrical energy, thereby reducing pumping power costs. The hydro-turbines may be converted to raw-water pumps, reversing the direction of water flow.

Right, Chapman 30-inch discharge cone valve, one of the countless Chapman Valves used throughout this system.

Big and modern as it is, the Water System of The District of Columbia is still expanding. They have already invested $85,323,559 in this public service and more funds have been appropriated for extra facilities. These include construction of additional filters, a new chemical building, and an added two story flocculation-sedimentation basin.

The source of water for the entire system is the Potomac River. Water is pumped through miles of conduits to the McMillan, Georgetown and Dalecarlia reservoirs for storage and subsequent filtration and treatment. Pumping stations are operated at McMillan, Dalecarlia, Bryant Street, Reno and Anacosta. Filtration plants are at McMillan and Dalecarlia. And throughout the entire system you'll find countless Chapman Valves. Chapman has been meeting the demands of this modern system for a number of years. The United States Army Corps of Engineers and the District of Columbia, Department of Sanitary Engineering insist that their valves meet every modern requirement . . . requirements that involve operation, size, pressure, stress and flow.



The **CHAPMAN** *Valve Manufacturing Co.*

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For more than 75 aggressive years, Chapman has been producing valves that meet every modern requirement. Chapman has the engineers, the metallurgists, the foundries, the manufacturing and testing facilities to meet the higher-pressure, higher-temperature demands of both today and tomorrow. At Chapman, there is always something new. It will pay you to talk with one of our engineers and let him bring you up to the instant. Write to us and he'll call at your convenience.

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Sea water rights are suddenly beginning to loom as a problem, even before water utilities are ready to start appropriating or riparianating the stuff for public supply, and the competitor for the raw material is a pretty potent one—the automotive industry. Clue to its interest in the briny was given by Earl D. Johnson, executive vice-president of General Dynamics Corp., when he stated that the use of sea water as a source of automotive fuel was being explored by GD—General Dynamics, of course—scientists who are "probing the possibilities for this revolutionary change." We might let the fact that the same amount of energy provided by the 900 gal of gasoline that the average American car owner uses each year is available in just 3 gal of sea water lull us, but, knowing these GD—still General Dynamics—people and looking into space, we're not at all sure we shouldn't hit the beach right now!

A new salt product designed especially for industrial and home automatic water softeners with separate brine tanks has been announced by International Salt Co. "Sterling Brine Kubes," 50-lb compressed cubes of pure salt, are said to be 100 per cent soluble, assuring clean brine tanks at all times. They are easy to store and handle. Compressed under tremendous pressure, each "cube" of pure salt takes up a space of $8 \times 8 \times 11\frac{1}{2}$ in. Some brine tanks are designed to take 6–8 such cubes, thus reducing the frequency of salt handling.

Pella, world capital during the days of Alexander the Great and Aristotle, has just recently been found by archeologists excavating some 24 mi northwest of Salonika, Greece. It was in 334 BC that Alexander rode out of Pella to conquer the world and in 168 BC that the Romans destroyed it, which should give some idea of the age of the elaborate clay water and sewage lines that were discovered when the first structure, a building 164 ft wide and 300 ft long, was excavated last year. The word the Greeks had for it, by the way, was probably "modern conveniences."

George M. Haskew has retired as chief engineer of Plainfield-Union Water Co., Plainfield, N.J., after a long career of service. He is succeeded by Conrad W. O'Connell, formerly with New York Water Service Corp. and most recently general manager of Bailly Engineering Enterprises, Pasadena, Calif.

The doorknob orthodontistry technique is being used for the fourth time this winter to rid Lake Musconetcong of its weeds. First step in the procedure, which was undertaken the last week in November, was to drain the lake down to a level low enough to permit the remaining water to freeze solid. When the ice is set, additional water will be admitted to the lake to float the ice and rip the weeds out by the roots—naturally. Then, of course, they should be put under the pillow of the New Jersey Dept. of Conservation & Economic Development, which will wake up to find itself richer by a good many dimes for the operation.

Raymond E. Melampy has been appointed safety engineer for Armco Drainage & Metal Products, Inc., Middletown, Ohio. He has been with Armco since 1940.

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Fine in '59 is the economic weather report according to a consensus of 212 economists polled by F. W. Dodge Corp. The economists are far more optimistic about the business outlook now than they were this time last year, and they are also much more nearly unanimous in their opinions on major economic indicators than they have been in the past. All but two of the 212 expect the gross national product to rise this year above its mid-1958 level, and all but four think industrial production will show a similar trend. On the average, they expect the gross national product to reach an annual rate of $460 billion by the fourth quarter of 1959, a rise of about 4.5 per cent during the year. Similarly, they expect the Federal Reserve index of industrial production to reach 147 by December 1959, going up about 5 per cent during the year.

In general, the economists feel that inflationary tendencies will continue, with some speedup in price rises toward the end of 1959. The median forecast is that the government's consumer price index will reach 125.5 by the end of 1959, as compared with the most recently reported figure of 123.7. Although the economists clearly think 1959 will be a good year, they recognize that there are soft spots to be bolstered and pitfalls to be avoided. Among those most frequently mentioned as possibilities are: inflation and runaway boom, with a counteraction some time after 1959; cutting off of the recovery by excessive credit

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restrictions; a relatively slow decline in the number of unemployed; a continuing profit squeeze facing many businesses; constantly rising wage rates; rising government deficits, high taxes, and a restrictive tax structure.

Portland Cement Assn. has named Cris Dobbins, president of Ideal Cement Co., Denver, chairman of the board. He succeeds George E. Warren, who has held the post during the past 2 years.

William P. Kliment, engineer of standards, Crane Co., Chicago, has been awarded ASA's Standards Medal for his "indefatigable efforts and outstanding achievements in the practical development and application of voluntary standards."

William Hunter Owen, formerly assistant director of the Div. of Sanitary Engineering, Tennessee Dept. of Public Health, has become associated with the consulting firm of Barge, Waggoner & Sumner, Nashville.

Boyd A. Bennett, former president of Northeastern Water & Electric Corp. of Philadelphia and an executive of South Bay Water Co. in charge of a number of water plants on Long Island, N.Y., died in Clearwater, Fla., Nov. 7, 1958. He was 73. Mr. Bennett was city manager of Clearwater from 1947 to 1951. Previously he had been city manager of Charlottesville, Va., and director of public works at Lynchburg, Va. He had been an AWWA member since 1937.

what price water?

WHAT PRICE WATER? is a 12-page, 4x9-in. booklet that calls to the attention of the reader the real value of public water supply. Comparing 1940 prices of a number of other common items with today's, it also presents a comparison of the 1940 costs of water works facilities with present ones as an indication that water rates must be boosted. Designed to fit in a standard No. 10 business envelope, the booklet sells at prices ranging from 10¢ to 2½¢ per copy. Imprints of your name and address as well as your rate per thousand gallons or cubic feet can be provided on lots of 500 or more. Ask for sample.

Willing Water Jewelry

Since their introduction in 1954, the Willing Water jewelry items have been most popular as awards, gifts, and good will builders. Presenting a blue-faced Willie in full stride, the emblem has made a hit as a lapel button as well as a decoration on a number of jewelry items. Included in the list of items now available are:

AW-1	Lapel Emblem (screw-back)	$ .75
AW-2	Lapel Pin (joint pin & safety catch)	.75
AW-3	Key Chain (spiral mesh chain and spring-lock holder)	1.25
AW-5	Zippo Cigarette Lighter (brush finish, boxed)	3.15
AW-8	Shortie Tie Clip (rhodium plated alligator clip)	$1.85
AW-9	Money Clip (rhodium plated)	2.75
AW-10	Cuff Links (rhodium plated disks)	4.50
AW-11	Earrings (screw type)	2.50

Willing Water Service Buttons

The popularity of Willing Water lapel emblems has led to the design of a special button for recognition of tenure. The design shown at the left was prepared on the request of the Alliance, Ohio, Water Department, which now uses the service buttons. It is now available to you, with your company name engraved on it. Minimum order is 25 buttons or pins. On such an order a die charge of $17.50 is made for inserting your company name. The pins (with joint pin and safety catch) or buttons (with screw-back) are priced as follows:

Bronze	$1.00
Sterling	1.50
Gold filled	2.00
10K Gold	$5.25
14K Gold	6.75

The minimum quantity (25) may be assorted as to years of service and as to metal used. Please specify clearly the number of each type required with pins (for female employees) and with screw backs.

AMERICAN WATER WORKS ASSOCIATION

2 Park Avenue **New York 16, N.Y.**

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Pipe, Cast Iron (and Fittings):
Alabama Pipe Co.
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Concrete Pressure Pipe Assn.
American Pipe & Construction Co.
Lock Joint Pipe Co.
Vulcan Materials Co.

Pipe, Copper:
American Brass Co.

Pipe, Steel:
Alco Products, Inc.
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.
Morgan Steel Products, Inc.

Pipe Cleaning Services:
National Water Main Cleaning Co.

Pipe Coatings and Linings:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div.
Reilly Tar & Chemical Corp.

Pipe Cutters:
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
Precision Chemical Pump Corp.
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.
Peerless Pump Div.

Pumps, Diaphragm:
Dorr-Oliver Inc.
W. S. Rockwell Co.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Peerless Pump Div.
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Fiese & Firstenberger
Layne & Bowler, Inc.
Peerless Pump Div.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Bailey Meter Co.
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Simplex Valve & Meter Co.
Sparling Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves: see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Infilco Inc.
Permutit Co.

Sodium Chloride:
International Salt Co., Inc.

Sodium Fluoride:
American Agricultural Chemical Co.

Sodium Hexametaphosphate:
Calgon Co.

Sodium Hypochlorite:
John Wiley Jones Co.
Wallace & Tiernan Inc.

Sodium Silicate:
Philadelphia Quartz Co.

Sodium Silicofluoride:
American Agricultural Chemical Co.
Tennessee Corp.

Softeners:
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon Co.
General Filter Co.
Industrial Chemicals, Inc.
International Salt Co., Inc.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.

Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Steel Plate Construction:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
Mueller Co.

Storage Tanks: see Tanks

Strainers, Suction:
James B. Clow & Sons
R. D. Wood Co.

Surface Wash Equipment:
Permutit Co.

Swimming Pool Sterilization:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Omega Machine Co. (Div., B-I-F Industries, Inc.)
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Tanks, Steel:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
General Filter Co.
Graver Water Conditioning Co.
Industrial Chemical Sales Div.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Turbines, Water:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valves, Altitude:
Golden-Anderson Valve Specialty Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co., Inc.
S. Morgan Smith Co.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Corp.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Regulating:
DeZurik Corp.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co.
S. Morgan Smith Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Simplex Valve & Meter Co.

Waterproofing:
Inertol Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
LaMotte Chem. Products Co.
W. A. Taylor & Co.
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite: see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1957 AWWA Directory.

here's what "out of sight out of mind" does to a water main

"Out of sight—out of mind" can be a mighty expensive philosophy in any water distribution system. The above unretouched photograph proves this point. It shows a badly tuberculated *eight inch* main whose *inside diameter was reduced to an average of almost 4.5 inches.* Resultant higher pumping costs with reduced pressure and carrying capacity make it costly to tolerate such conditions. That is why the savings effected in reduced pumping costs frequently pay for the *low cost* of National water main cleaning.

Since there's never a charge or obligation to inspect your mains, *call National now!*

Call in National today!

NATIONAL WATER MAIN CLEANING COMPANY

50 Church Street • New York, N. Y.

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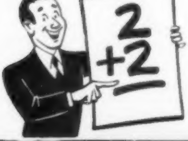
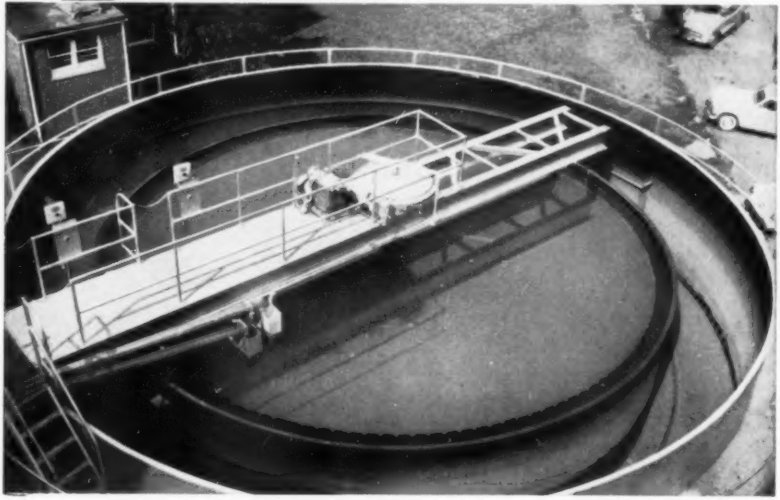


The solution to this problem is always the same . . . but

Water Treatment Problems are different

No two water treatment problems are exactly alike. The right solution to each can only be arrived at after a careful study of the local conditions. Variables such as raw water composition, rate of flow and results required automatically rule out the cure-all approach. The installation shown below is a good example of how equipment should be selected to fit the job . . . and not vice versa.



Fountain City
TENNESSEE
PeriFilter System employs split filter for continuous operation

Close up of PeriFilter System taken while backwashing right side of filter. Left side of filter remains in operation.

Consulting Engineers: Polk, Powell and Hendon, Birmingham, Alabama.

Producing 1.0 MGD of finished water from limestone springs at Fountain City, this Dorrco PeriFilter System consists of a single 30′ dia. Hydro-Treator surrounded by an annular rapid sand filter. To maintain continuous operation, the filter is split by a partition plate and backwashed one half at a time. During backwashing, Hydro-Treator effluent overflows into the inner launder and is distributed to the opposite half of the filter. The results at Fountain City have been uniformly excellent with an average turbidity in the filtered water of less than 0.3 ppm.

For more information on the complete line of D-O equipment for the water works industry write for a copy of Bulletin No. 9041. Dorr-Oliver Incorporated, Stamford, Connecticut.

Hydro-Treator, PeriFilter, T.M. Reg. U. S. Pat. Off.

Every day over 8½ billion gallons of water are treated by Dorr-Oliver equipment.

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